Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . SEPTEMBER-OCTOBER 1936

A Study of Reinforcement in Concrete Slabs*

By Inge Lyse†
MEMBER AMERICAN CONCRETE INSTITUTE
and George R. Wernisch‡

SYNOPSIS

The results of an investigation of the use of different types and grades of reinforcing steels in 4.0 in. (nominal effective depth 3.0 in.) thick and 34-in. wide concrete slabs of various lengths, in connection with different strengths of concrete, are presented in this paper.

It was found that the full yield-point value of the reinforcement would be developed in steels up to 95,000 p. s. i. (the maximum used in this study) when flexure governed failure; increasing the strength of the concrete did not materially increase the effectiveness of the reinforcement; the reinforcement was as effective in sixteen-foot spans as in five-foot spans; up to a spacing of seventeen inches and bar size of one inch square (the maximum size and spacing used in this investigation), size and spacing of reinforcement did not influence the strength of the slabs; ordinary deformed reinforcing bars were as effective as welded wire fabric; and when reinforcement was stressed to 30,000 p. s. i. only hairline cracks appeared in the slabs, and these cracks wholly disappeared when the loads were removed.

INTRODUCTION

Proposed building regulations for reinforced concrete contemplate an allowable working stress of one-half the yield point of the steel for welded wire fabric or other steel reinforcement not exceeding ¼ in. in diameter, with a maximum of 30,000 p. s. i. Furthermore, the spacing of principal reinforcement is limited in most building codes to three

*Received by the Institute Secretary, June 17, 1936.
†Research Associate Professor of Engineering Materials, in charge of Frits Engineering Lab., Lehigh University, Bethlehem, Pa.
‡Concrete Reinforcing Steel Institute Research Fellow, Lehigh University, Bethlehem, Pa., in immediate charge of slab investigation.

times the depth of the slab. Very little data are presented to justify these limitations.

The one-way slab pilot investigation reported herein was undertaken in cooperation with the Concrete Reinforcing Steel Institute for the purpose of securing more data on the behavior of reinforced concrete slabs, studying the effect of high and low yield-points steels and rail steel, the size and spacing of reinforcing bars, the strength of concrete, the length of slabs, the appearance of initial cracks, and the strength and cracking characteristics of slabs reinforced with welded wire fabric as well as with ordinary deformed bars.

TEST PROGRAM

The program was so arranged that each variable could be studied independently. All slabs were 34 in. wide and had a nominal effective depth of 3.0 in.

In Group I, the effect of the yield point of the reinforcing steel was studied. The yield point varied from 46,000 to 95,000 p. s. i.

The effect of the strength of concrete was studied in Group II. The concrete strength varied from 2640 to 5850 p. s. i.

Group III was designed to study the effect of the length of span to determine whether the effectiveness of the steel in resisting moment was materially influenced by the length of span. The lengths of slabs varied from five to sixteen feet, simply supported, and five and eight feet, continuous.

In Group IV the effect of the size and spacing of bars was studied, the steel areas being kept approximately constant for each length of slab. The size of bars varied from ¼ in. round to 1 in. square, and the spacing varied from 2 to 17 in. Three of the slabs were reinforced with welded wire fabric 0.2 to 0.26 in. in diameter for comparison with slabs reinforced with deformed bars. Table 1 shows the details of the test program.

MATERIALS, MANUFACTURE AND TESTING

All the materials used in the investigation, except the welded wire fabric, which was purchased on the market, were donated, the fine aggregate and all the reinforcing bars, except the rail steel, by the Bethlehem Steel Co., the rail steel by the Rail Steel Bar Association, and the cement by the Lehigh Portland Cement Co.

All the slabs were poured in steel forms and permitted to remain therein for three days, after which they were placed in the moist room until the age of 28 days, at which time they were tested. The concrete test cylinders were stored in a similar manner.

TABLE 1—TEST PROGRAM

Spec. No.	Concrete Strength p. s. i.	Y. P. of Steel p. s. i.	Span Length ft.	Bars			Continuity Condition
				No.	Size in.	Per Cent Approx.	
GROUP I—EFFECT OF YIELD POINT STEEL							
9	2890	46,000	8	7	¼ d	0.35	Simple*
10	2795	55,000	8	7	¼ d	0.35	Simple
11	2600	75,000	8	7	¼ d	0.35	Simple
12	3390	95,500	8	7	¼ d	0.35	Simple
29	5260	93,000	8	7	⅜ d	0.77	Simple
GROUP II—EFFECT OF STRENGTH OF CONCRETE							
13	5500	44,300	8	7	⅜ d	0.77	Continuous†
14	5500	93,000	8	7	⅜ d	0.77	Continuous
15	2900	49,300	8	8	⅜ d	2.48	Continuous
15a	2700	47,000	8	6	⅜ d	1.86	Continuous
16	5840	50,000	8	8	⅜ d	2.48	Continuous
16a	5850	47,000	8	6	⅜ d	1.86	Continuous
17	2800	46,000	5	6	½ d	1.20	Continuous
18	2640	48,000	8	6	½ d	1.20	Continuous
32	5750	50,000	8	7	⅜ d	0.77	Simple
6	2780	47,500	8	7	⅜ d	0.77	Simple
GROUP III—EFFECT OF SPAN LENGTH							
1a 1b 1c	2990 No Steel 2907		5	No Steel		0	Simple
2	2786	47,000	5	6	½ d	1.20	Simple
3	2700	46,000	8	6	½ d	1.20	Simple
25	3085	47,000	12	6	½ d	1.20	Simple
28	3200	38,950	16	2	1 sq.	2.00	Simple
17	2800	46,000	5	6	½ d	1.20	Continuous
18	2640	48,000	8	6	½ d	1.20	Continuous
13	5500	44,300	8	7	⅜ d	0.77	Continuous
14	5500	93,000	8	7	⅜ d	0.77	Continuous
GROUP IV—EFFECT OF SPACING AND SIZE OF REINFORCEMENT							
30‡	2900	58,100**	5	7	0.2 d	0.20	Simple
4‡	2640	62,900**	8	17	0.2 d	0.56	Simple
5	2690	56,600	8	15	¼ d	0.75	Simple
6	2784	47,500	8	7	⅜ d	0.77	Simple
7	2640	50,000	8	4	½ d	0.80	Simple
20‡	2750	60,500**	12	17	0.26 d	0.92	Simple
26	3000	50,000	8	5	½ d	1.00	Simple
21	3100	53,100	12	13	⅜ d	1.43	Simple
22	3140	48,500	12	7	½ d	1.40	Simple
23	3080	47,100	12	5	⅝ d	1.55	Simple
24	3110	50,300	12	3	¾ d	1.32	Simple
31	2880	47,900	16	8	½ d	1.60	Simple
28	3200	38,950	16	2	1 sq.	2.00	Simple

*Unrestrained slab, two equal loads placed at quarter points.
†Four point, equal loading, where moment at the supports equals twice the moment at the center of span, for live load only.
‡Slabs reinforced with welded wire fabric.
**Elastic limit = 80 per cent ultimate stress.
Note: Slab 1b. broke during handling at age of two days.

Nine 3 by 6-in. concrete control cylinders were made with each slab, and five pieces of reinforcement were taken from each slab and subjected to tensile tests to determine the yield point of the steel.

Two types of loading were applied in testing: two-point loading (referred to hereafter as simple loading) and the four-point loading (referred to as continuous loading). The simple loading was used for

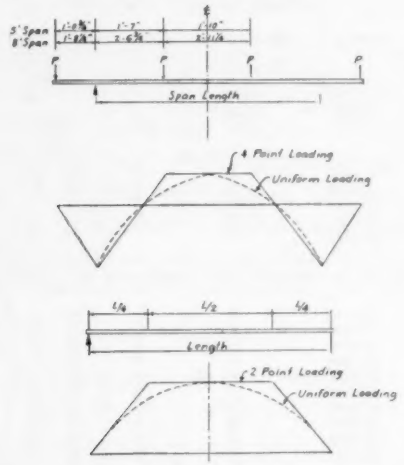


FIG. 1—LIVE LOAD BENDING MOMENT DIAGRAMS FOR CONTINUOUS AND SIMPLE LOADINGS

the slabs tested in an unrestrained position; two equal loads were placed at the quarter points of the slab. This type of loading simulated uniform load behavior. All simply loaded slabs were tested in an upside down position in order to facilitate the crack study (the slabs were whitewashed to aid in detection of initial cracks). Concentrated loads were used instead of uniform loads because they eliminated all possibility of arch action and because of a simpler testing arrangement. Fig. 1 illustrates the loading arrangements for both types of tests.

The slabs which were tested as continuous were subjected to four concentrated loads, shown in Fig. 1 and also in Fig. 2, placed in such a manner that the moment at the support was equal to twice the center moment.

Deflection observations were taken by placing nine Ames dials (reading to one-thousandths of an inch) in three rows, one row along each edge and one row down the center, at the quarter points and at the center of the slab (refer to Fig. 3). After several slabs were tested, it was found that the deflections were very nearly constant across the entire width of slab, and thereafter only one row of dials was placed along the center of the slab, longitudinally.

After the Ames dials were removed, approximate deflection readings were taken with a scale graduated to 1/100 in.

A summary of the results of the tests of all the slabs is given in Table 2.

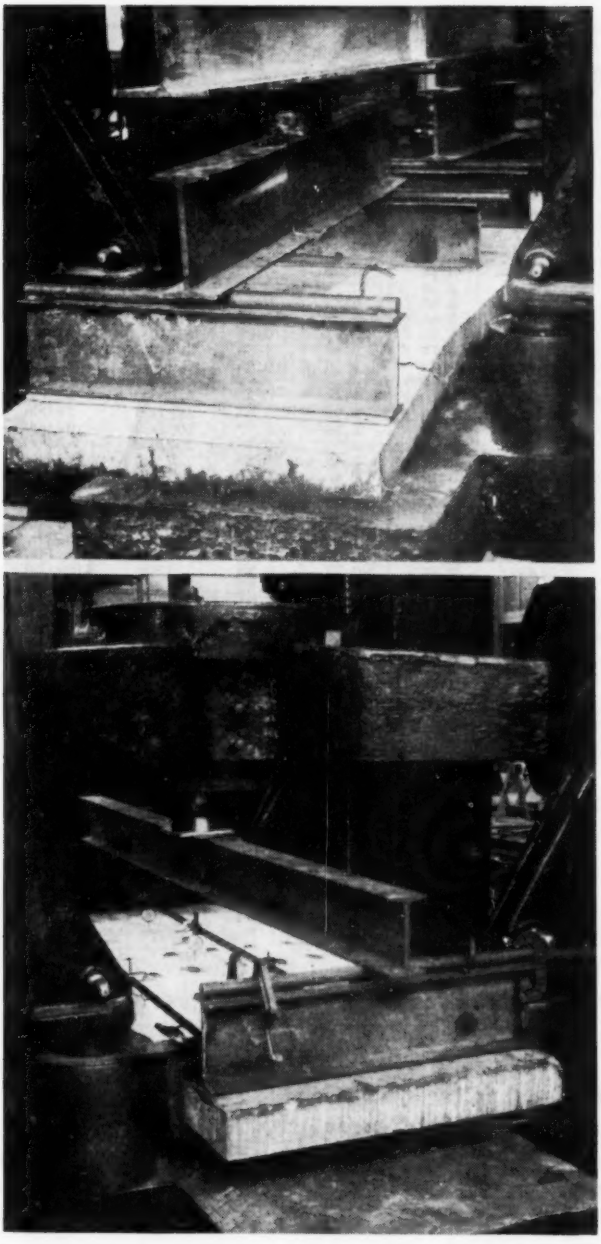


FIG. 2—CONTINUOUS LOADING

FIG. 3—SIMPLE LOADING, DEFLECTION DIALS

TABLE 2—SUMMARY OF RESULTS

Slab No.	Span ft.	Average Thickness in.	Eff. "D" in.	Steel Area sq. in.	"J"	Total Res. Moment in.-lb.	Calc. Steel Stress p. s. i.	Yield Point Steel p. s. i.
1a	5	4	——	——	——	53,000	——	——
1c	5	4	——	——	——	47,000	——	——
2	5	4¼	3.12	1.20	0.87	172,600	53,000	47,000
3	8	4⅛	3.06	1.20	0.87	165,300	51,800	46,000
4*	8	4⅛	3.00	0.56	0.91	142,300	93,100	62,900‡
5	8	4⅛	3.08	0.75	0.90	153,800	74,000	56,600
6	8	4⅛	3.00	0.77	0.89	143,300	69,800	47,500
7	8	4⅛	3.12	0.80	0.89	155,300	70,000	50,000
9	8	4⅛	3.10	0.35	0.91	72,500	73,400	46,000
10	8	4¼	3.40	0.35	0.91	88,700	81,800	55,000
11	8	4⅛	3.06	0.35	0.91	104,500	107,200	75,000
12	8	4⅛	3.00	0.35	0.91	120,000	125,800	95,500
13†	8	4⅛	3.00	0.77	0.89	146,200	71,100	44,300
14†	8	4⅛	2.88	0.77	0.89	272,400	138,000	93,000
15†	8	4⅛	2.75	2.48	0.83	305,400	53,800**	49,300
15a†	8	4	2.88	1.86	0.85	260,000	57,200	47,000
16†	8	4⅛	3.00**	——	——	——	——	——
16a†	8	4⅛	3.00	1.86	0.85	260,400	55,000	47,225
17†	5	4⅛	3.00	1.20	0.87	222,000	71,000	46,000
18†	8	4⅛	2.82	1.20	0.87	200,000	67,900	47,000
20*	12	4⅛	3.00	0.92	0.88	204,400	87,200	60,500‡
21	12	4⅛	3.00	1.43	0.87	225,400	60,300	53,100
22	12	4⅛	3.10	1.40	0.87	206,700	54,750	48,500
23	12	4⅛	3.06	1.55	0.86	209,400	51,400	47,100
24	12	4⅛	2.94	1.32	0.87	198,900	59,000	50,300
25	12	4⅛	3.06	1.20	0.87	180,400	56,400	47,000
26	8	4⅛	3.00	1.00	0.88	181,300	68,700	50,000
28	16	4	2.75	2.00	0.85	205,750	43,900	38,950
29	8	4½	2.94	0.77	0.89	219,300	109,000	93,000
30*	5	4⅛	2.89	0.20	0.92	56,600	106,600	58,100‡
31a††	4	4⅛	3.00	0.35	0.91	146,800	153,700	95,500
31	16	3½	2.94	1.60	0.85	218,000	54,300	47,900
32	8	4¼	3.12	0.77	0.89	147,500	69,000	50,000

*Welded wire fabric reinforcement.
†Continuous.
‡Elastic limit = 80 per cent ultimate stress.
**Diagonal tension failure outside support.
††Concentrated load at center.

EFFECT OF YIELD POINT OF STEEL

Fig. 4 indicates the relationship between the actual and calculated resisting moments developed in the slabs at time of failure and the yield point of the reinforcement. The calculated resisting moment was obtained by means of the straight line theory, assuming no tension in the concrete and the reinforcement effective to its yield point. It is to be observed that all the slabs developed a resisting moment in excess of the calculated moment.

In the case of continuous slabs, the excess resisting moment was greater than that in the simply loaded slabs, probably due to the fact that the bending moment curve over the supports in the continuous slab has a tendency to flatten, rather than come to an abrupt point.

Deflection is a matter that is certain to receive special attention with the use of high yield-point steels. Referring to Fig. 5, it is apparent that the deflections of slabs containing equal steel areas, regardless of the yield point of the steel, are approximately equal for equal steel stresses within the yield-point value. Considering the case

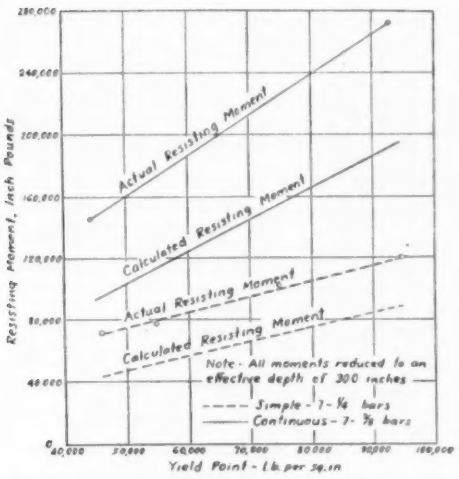

Resisting Moment, Inch Pounds
Actual Resisting Moment
Calculated Resisting Moment
Actual Resisting Moment
Calculated Resisting Moment
Note - All moments reduced to an effective depth of 300 inches
Simple - 7 - ¼ bars
Continuous - 7 - ⅜ bars
Yield Point - Lb. per Sq. in.

FIG. 4—EFFECT OF YIELD POINT STEEL ON RESISTING BENDING MOMENT

of simply supported slabs, which is a condition seldom realized in building construction, the deflection of the slab containing the 56,600 p. s. i. Y. P. steel, at fifty per cent of its yield-point value, is approximately 0.15 in., whereas the deflection of the slab containing the 93,000 p. s. i. Y. P. steel is approximately 0.29 in. at fifty per cent of its yield-point value. The deflections of the continuous slabs are much smaller, being 0.05 in. at a steel stress of fifty per cent of the yield-point value of the 43,300 p. s. i. Y. P. steel slab, and 0.13 in. at fifty per cent of the yield-point value of the slab containing the 93,000 p. s. i. Y. P. steel. In terms of span length the deflection of the continuous slab containing the 93,000 p. s. i. Y. P. reinforcement, at half its yield-point value, is 0.13/96 = 1/739th of the span length. It seems, therefore, that deflections, should not cause much concern in slabs containing high yield-point reinforcement.

The question of cracking will be mentioned here briefly. Table 3 gives an indication of the cracking behavior of slabs containing high yield-point steel.

TABLE 3

Slab	Y. P. Steel p. s. i.	Calculated Steel Stress at:		Steel Area sq. in.	Type of Loading
		First Crack p. s. i.	0.01" Crack p. s. i.		
14	93,000	40,700	62,300	0.77	Continuous
29	93,000	33,600	63,600	0.77	Simple

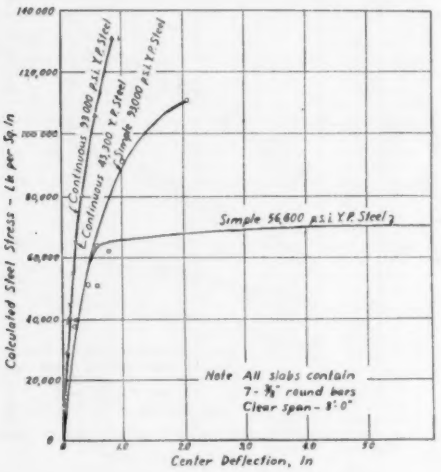


FIG. 5—STRESS-DEFLECTION CURVES FOR CONTINUOUS AND SIMPLE SPANS

The cracking at fifty per cent of the yield-point of high yield-point steels did not appear to be important, the cracks being infrequent and hairline at these stresses.

Generally, any type or grade of reinforcing steel with yield-point stress up to 95,500 p. s. i. developed its full yield-point value in flexure. Stressing the high yield-point steels to fifty per cent of their yield-point value did not cause excessive deflections nor cracking in the slabs.

EFFECT OF STRENGTH OF CONCRETE

All slabs gave strengths considerably in excess of the calculated strengths. Slabs No. 13 and 14 (of equal steel areas) made with 5500 p. s. i. concrete, had an average excess strength over that calculated (by assuming that the steel was effective only to its yield-point) of 70, 750 in.-lb. Slabs No. 17 and 18, with an average concrete compressive strength of 2720 p. s. i., had an average excess strength of 70,200 in.-lb. Slab No. 15a, with a concrete compressive strength of 2700 p. s. i. had an excess strength of 41,500 in.-lb., whereas slab No. 16a, with a concrete compressive strength of 5850 p. s. i. had an excess strength of 37,200 in.-lb. In all cases of comparison, the slabs of low concrete strength had an excess resisting moment approximately equal to that of slabs made of high strength concrete. The aforementioned slabs were all subjected to continuous loading. Slabs No. 13, 14, 17 and 18 had an excess strength considerably greater than slabs

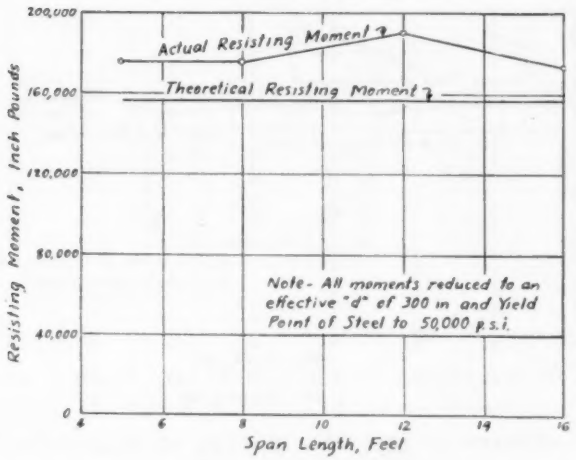


FIG. 6—EFFECT OF SPAN LENGTH ON RESISTING MOMENT OF SLABS

15a and 16a, probably due to the much smaller steel areas contained in the first four slabs mentioned.

Slabs No. 6 and 32, simply supported, with concrete strengths of 2780 and 5750 p. s. i. respectively, had excess strengths of 45,800 and 40,060 in.-lb. respectively.

It appears that the steel reinforcement was as effective in slabs of low concrete strength as in slabs of high concrete strength, provided the slab is designed in accordance with present methods, using theoretically balanced reinforcement as maximum percentage of steel.

EFFECT OF SPAN LENGTH

Fig. 6, giving the relationship between span length and resisting bending moment, indicates that the reinforcing steel was as effective in long spans as in short spans. All moments were adjusted to correspond to an effective depth of 3.0 in. and a yield-point of steel of 50,000 p. s. i. In the case of the simply supported slabs, the excess resisting moment developed in the 5-ft. slab was 19,850 in.-lb., whereas that of the 16-ft. slab was 23,600 in.-lb. All four simply supported slabs in this series had an average excess moment of 22,040 in.-lb., with a minimum excess of 19,850 in.-lb. for the 5-ft. slab and a maximum excess of 27,500 in.-lb. for the 12-ft. slab. In terms of percentage of total resisting moment, the 5-ft. slab had an excess of 11.5 per cent and the 16-ft. slab an excess of 11.45 per cent.

Span length, therefore, caused no reduction in the effectiveness of the reinforcing steel up to a length of sixteen feet, the maximum used in this investigation.

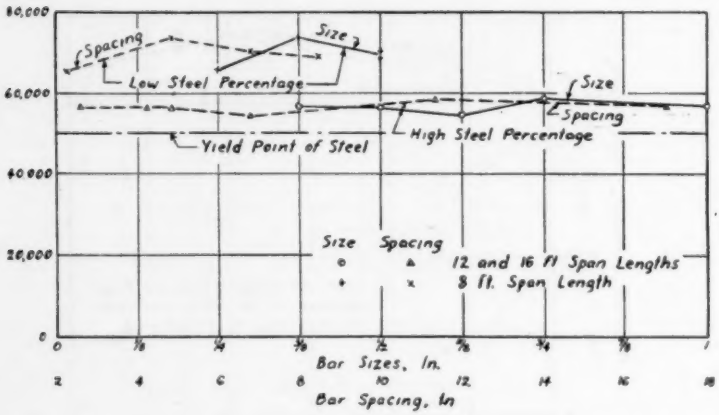


FIG. 7—EFFECT OF SIZE AND SPACING OF REINFORCING BARS

EFFECT OF SIZE AND SPACING OF REINFORCEMENT

In Fig. 7, the size and spacing of reinforcing bars are plotted against the calculated stress in the steel at the failure of the slabs. The stresses have been adjusted to correspond to a uniform yield-point of steel of 50,000 p. s. i. From this diagram it appears that the size and spacing of the reinforcing bars did not influence the strength of the slab.

In the 8-ft. series, slab No. 5, with ¼-in. bars spaced 2.27 in. on centers, had an excess strength of 36,000 in.-lb., whereas slab No. 6, containing ⅜-in. bars spaced 4.87 in. on centers, had an excess strength of 45,800 in.-lb. The excess strength of the slab reinforced with ½-in. bars on 8.50 in. centers was 44,400 in.-lb. Here the larger bars and spacings gave slightly higher results than the smaller bars and spacings.

In the 12-ft. series, slab No. 21, reinforced with ⅜-in. bars, spaced 2.61 in. on centers, had an excess strength of 26,800 in.-lb., whereas slab No. 24, with ¾-in. bars spaced 11.35 in. on centers, had an excess strength of 29,200 in.-lb.

The excess strength in slab No. 28, containing two 1-in. square bars spaced 17 in. on centers, was 23,600 in.-lb., indicating that even a 17 in. spacing in a 16 ft. length did not materially influence the strength of the slab.

Slab No. 28, slabs No. 21 to 24 inclusive, and slabs No. 4 and 20 (containing wire fabric reinforcement) were studied to determine whether the size and spacing of the reinforcement bars influenced the width and number of cracks, and the distribution thereof. The data are presented in Table 4.

TABLE 4

Slab No.	Y. P. Steel p. s. i.	Bars		Area sq. in.	Calculated Steel Stress at:		No. of Cracks	Average Distance Between Cracks inches
		No.	Size in.		First Crack p. s. i.	0.01" Crack p. s. i.		
4	78,600*	17	0.2 d	0.56	32,100	39,600	13	7.4
20	75,500*	17	0.26 d	0.92	21,600	32,500†	16	9.0
21	53,100	13	3/8	1.43	21,900	51,300	26	5.5
22	48,500	7	1/2	1.40	17,800	43,000	27	5.3
23	49,100	5	5/8	1.56	16,500	47,200	22	6.5
24	50,300	3	3/4	1.32	22,200	46,700	23	6.3
28	38,950	2	1 sq.	2.00	10,700	30,500	31	6.2

*Ultimate strength of welded wire fabric.
†Crack of 0.015 in. in width.

The data indicate that the width and distribution of cracks were not affected by the size and spacing of the reinforcement.

INITIAL CRACKING

The data indicate that, generally, the initial cracks in the reinforced slabs did not appear before the flexural stress in the concrete had reached a value approximately equal to the strength of the unreinforced concrete slabs, and that the initial cracking in the five high strength concrete slabs (5500—5800 p. s. i.) occurred at a concrete flexural stress approximately one-third greater than in slabs of low concrete strength (2600—3200 p. s. i.).

The exactness of the load at incipient cracking is somewhat doubtful, due to the difficulty involved in observation. Probably if an accurate method of observing initial cracking could be devised, the cracking stress in the concrete would correspond very closely to the ultimate strength of the unreinforced slab, plus the assistance rendered by the reinforcement.

The stresses in the steel computed on the basis of no tension in the concrete, at initial cracking, are of little significance since these stresses depend on the amount of reinforcement in the slab. The steel stresses in slabs 5, 6 and 7, containing approximately equal amounts of steel, were approximately equal for initial cracking. Slabs 21 to 24 inclusive, indicate the same approximate relationship. The ratio of the average steel stresses of the aforementioned groups (1.84) is approximately equal to the ratios of their average area (1.72). The average concrete cracking stresses for both the aforementioned groups, as shown in Table 5, were approximately equal.

Before cracking, the slabs act very much like homogeneous material, and the steel is stressed n times the stress in the concrete at the plane of the reinforcement. The calculated stress in the steel just *before* cracking, on this basis, is thus found to be small. Immediately *after*

TABLE 5

Slab No.	Length of Span ft.	Bars		Total Steel Area sq. in.	Calculated Steel Stress at First Crack p. s. i.	Concrete Flexural Stress at First Crack p. s. i.	Average Concrete Stress p. s. i.
		No.	Size in.				
5	8	17	¼	0.75	28,900	580	—
6	8	7	⅜	0.77	39,600	793	—
7	8	4	½	0.80	32,200	679	677
21	12	13	⅜	1.43	21,900	777	—
22	12	7	½	1.40	17,800	618	—
23	12	5	⅝	1.55	16,500	647	—
24	12	3	¾	1.32	22,200	693	684

cracking, assuming that the concrete does not resist tension, these stresses are greatly increased, not only in their calculated value but also in their observed value. Therefore the initial cracking caused the large stresses in the steel. The calculated steel stresses given in this paper are based on no tension in the concrete.

Table 6 shows the relationship between the steel stresses after cracking and the size of the cracks.

TABLE 6

Slab No.	Y. P. Steel p. s. i.	Calculated Steel Stress at:			Steel Area sq. in.	Type of Loading
		First crack p. s. i.	0.01″ crack p. s. i.	0.02″ crack p. s. i.		
9	55,000	43,000	60,700	70,200	0.35	Simple
11	75,000	47,500	60,500	80,000	0.35	Simple
12	95,500	25,700	80,500	104,000	0.35	Simple
13	40,000	39,700	42,300	47,400	0.77	Continuous
14	93,000	40,700	62,300	80,200	0.77	Continuous
15A	49,300	17,600	35,300	—	1.86	Continuous
16	50,000	20,200	54,000	—	2.48	Continuous

Several slabs were subjected to loads equivalent to approximately eighty per cent of their calculated yield-point loads, and then unloaded, whereupon all the cracks disappeared. In the case of slab No. 5, containing steel with a yield-point of 56,000 p. s. i., the slab was loaded to a calculated steel stress of 46,000 p. s. i. and the load then removed. All of the fifteen cracks in the slab at that load, several 0.01 in. in width, disappeared.

EFFECTIVENESS OF TYPE OF REINFORCEMENT

Referring to the general results it is noted that slab No. 30, containing 0.20 sq. in. of welded wire fabric, developed a strength 84 per cent in excess of the calculated elastic limit value of the steel, and 49 per cent in excess of the ultimate strength value. Slab No. 4, containing 0.56 sq. in. of fabric developed a strength 18 per cent beyond the calculated ultimate strength value of the steel. Slab No. 20, con-

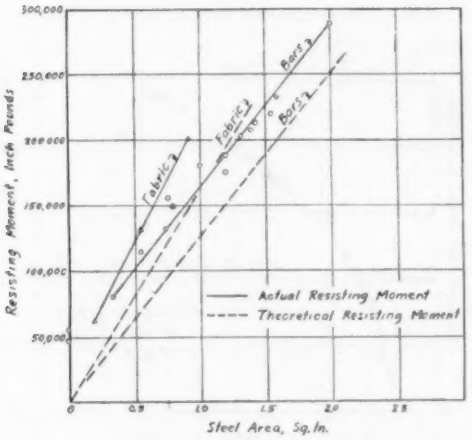


FIG. 8—EFFECT OF STEEL AREA ON RESISTING MOMENT

taining 0.92 sq. in. fabric, developed a strength 11 per cent in excess of the calculated ultimate strength value of the steel. It should be noted that as the steel areas increased in the slabs, the calculated steel failure stresses decreased.

Also, slab No. 9, reinforced with ¼-in. deformed bars, 0.35 sq. in. in area, developed an excess strength 60 per cent; slab No. 10, 49 per per cent; slab No. 11, 43 per cent; and slab No. 12, 32 per cent beyond their calculated values. Here it should be noted that the excess strength, in terms of the yield-point of the reinforcement, decreased as the yield-point increased.

A better understanding of the excess strength may be obtained from a study of the total resisting moments (see Fig. 8). For small percentages of reinforcement (and consequent low total resisting moments) the excess strength is seen to aid greatly in increasing the resistance of the slab, and thus increase the fictitiously calculated steel stresses tremendously. Slab No. 30, although it had an excess moment of only 25,700 in.-lb., developed steel failure stresses 84 per cent in excess of the elastic limit, whereas slab No. 4, with an excess moment of 46,000 in.-lb., developed steel failure stresses only 48 per cent in excess of the elastic limit, because the excess strength in slab No. 4 was absorbed by three times the steel area of that in slab No. 30.

In computing the excess resistance of slabs reinforced with welded wire fabric (see Fig. 8) the authors assumed that the fabric was effective only to its elastic limit (defined by A. S. T. M. as not less than 80 per cent of the ultimate stress). Considering that the stress-strain

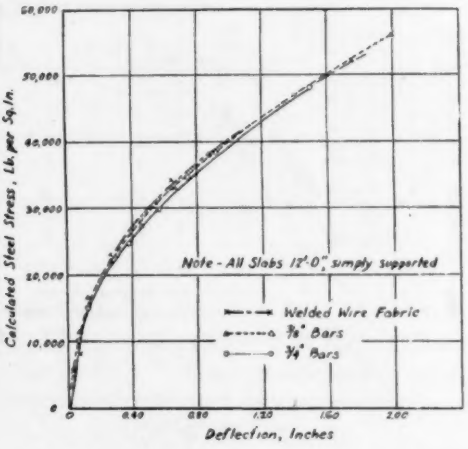


FIG. 9—EFFECT OF BAR SIZE ON DEFLECTIONS

curve for wire fabric does not have a plastic stage, it seems logical to assume that the fabric took load to its ultimate strength. The true value of the excess strength of slabs reinforced with welded wire fabric is therefore uncertain, but it may be considered somewhere between the extreme calculations; for slab No. 30, between 17,900 and 25,700 in.-lb., slab No. 4, between 21,700 and 46,000 in.-lb., and slab No. 20 between 20,400 and 56,000 in.-lb. Assuming that the excess strength will be more correct when the fabric is considered effective to its ultimate value, this excess was approximately equal to that of slabs reinforced with deformed bars.

It should be remembered that since the excess strength is practically constant for the different types and grades of reinforcement, the percentage of excess over the calculated resisting moment will decrease with the increase in the amount of reinforcement. This can be illustrated by comparing slabs No. 10 and 24. Slab No. 24 had a total resisting moment of 198,900 in.-lb. and an excess moment of 29,200 in.-lb. In terms of steel yield-point strength, the steel stress was increased 17 per cent by this excess moment. Slab No. 10, with a total resisting moment of only 88,700 in.-lb. and an excess moment of 29,200 in.-lb. had a calculated steel stress 49 per cent in excess of the yield-point. Thus it is observed that equal excess strengths in one case increased the stress only 17 per cent while in the other case it increased the stress 49 per cent. Calculated steel stresses, therefore, did not give direct indication as to the full strength of the slab and it seems that slabs reinforced with deformed bars were as effective in resisting bend-

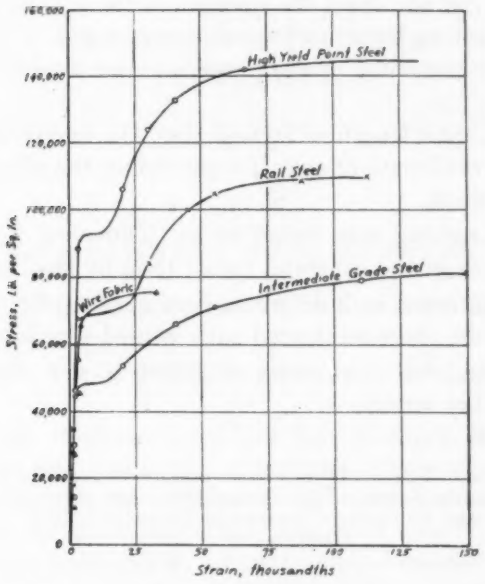


FIG. 10—TYPICAL STRESS-STRAIN CURVES FOR REINFORCEMENT

ing moments as slabs reinforced with welded wire fabric (refer to Fig. 8).

The excess resistance in the slabs probably is due to concrete contributory effect and increase in the j value of the slab near ultimate loads. Further investigation is necessary for the solution of this problem.

The deflection of slabs reinforced with welded wire fabric or deformed bars, were approximately equal for equal stresses, as can be seen from Fig. 9. Stress-strain diagrams for the different types of reinforcements used are shown in Fig. 10.

SUMMARY

The results of this investigation of one-way reinforced concrete slabs indicated that:

1. All grades of reinforcing steels, up to a maximum of 95,000 p. s. i., yield point (the maximum used) developed their full yield-point value in flexure.

2. The steel reinforcement developed its full resisting strength in both high and low strength concrete in slabs which were designed in accordance with present practice.

3. Spacing did not affect the strength of the slab up to a maximum spacing of seventeen inches and a maximum bar size of one square inch; these were the maximum spacing and bar sizes used in this investigation.

4. Up to a span length of sixteen feet, the maximum used in this investigation, the length of span did not reduce the effectiveness of the steel reinforcement.

5. Initial cracking was found to be influenced by the ultimate flexural strength of the concrete, rather than by steel stresses.

6. Slabs reinforced with deformed bars were as effective in resisting moments as were those reinforced with welded wire fabric.

7. At a calculated steel stress of 30,000 p. s. i. cracking and deflections were not serious.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for January-February 1937. Discussion should reach the Institute Secretary by December 1, 1936.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY 1937

Discussion of a paper by Lyse and Wernisch:

A Study of Reinforcement in Concrete Slabs*

K. HAJNAL-KONYI†

THE paper by Professor Lyse and Mr. Wernisch is a remarkable contribution to our knowledge about the effectiveness of different types of reinforcement in connection with different strengths of concrete. One of the most interesting results of these investigations is the fact that the resistance of all tested slabs exceeded the calculated value (by assuming that the steel was effective to its yield point), even with a reinforcement of 2.14 per cent (slab No. 28). The excess strength above the yield point varies between 9 per cent (No. 23) and 84 per cent (No. 30). "In all cases of comparison the slabs of low concrete strength had an excess resisting moment approximately equal to that of slabs made of high strength concrete." Although the phenomenon of the excess strength has been well known for many years, we have no satisfactory explanation for it. The paper concludes by saying that "further investigation is necessary for the solution of this problem" (p. 15).

It should be also mentioned that in the slabs with the higher percentages of reinforcement the calculated concrete stress exceeds considerably the compressive strength of the concrete. For instance in slab No. 21 with 1.40 per cent reinforcement

(d = 3.0 in., b = 34 in., A_s = 1.43 sq. in., j = 0.87, jd = 2.61 in., n = 1.17 in.)

$$c = \frac{225400 \times 2}{34 \times 2.61 \times 1.17} = 4340 \text{ p.s.i.}$$

as against the observed compressive strength of the concrete of only 3100 p.s.i., an excess of 40 per cent.

*JOURNAL, Amer. Concrete Inst., Sept.-Oct. 1936, p. 1.
†Consulting Engineer, London England.

May I direct attention to a paper by Professor Saliger (Vienna) recently published in *Beton u. Eisen.** Saliger gives an analysis of the stresses in a reinforced concrete beam between two cracks and develops a new theory of the distribution of stresses in the concrete and steel which seems to me to be a good explanation of the excess strength.

I want to show how this new theory can be used for the interpretation of the tests of Professor Lyse and Mr. Wernisch. For this purpose it is necessary to give an idea about the principles of Professor Saliger's theory.

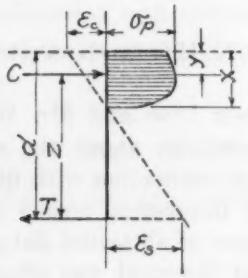


FIG. 1

Its basis is the fact that the distribution of the compressive stresses in the concrete before failure is not linear as assumed by the "classic" theory. Fig. 1 shows approximately the real distribution of the stresses and strains in a reinforced concrete section. The symbols, C, T, d, x, y, z do not need any further explanation, the signification of the other symbols is the following:

ϵ_c = strain of the concrete
ϵ_s = strain of the steel
σ_p = crushing strength of the concrete, tested in 4-sided prisms (which is about the same as tested in cylinders)

It is proved by many tests that ϵ_c can become much higher than the ultimate strain in prism or cylinder tests. The maximum strain of concrete prisms (tested in compression) varies between 2 and 4 per mille. In beams strains up to 10 per mille have been measured, strains between 3 per mille and 7 per mille are normal and $\epsilon_c = 5$ per mille can be assumed as a rough average. It is of interest that the stress of the concrete can decrease at the edge according to the slope of the stress-strain diagram of the concrete.

*Saliger: Bruchzustand und Sicherheit in Eisenbetonbalken. (Rupture and safety in reinforced concrete beams) *Beton u. Eisen* 1936. No. 19, 30.

If b denotes the width of the rectangular concrete section, we obtain

$$c = k \,.\, b \,.\, x \,.\, \sigma_p$$

k is a variable factor between 0.80 and 0.95 depending on the stress-strain diagram, it can be assumed to be 0.85 to 0.90. y varies between $0.4x$ and $0.5x$, it can be assumed to be $\frac{kx}{2}$. It is obvious that the "plastic" distribution of the concrete stresses as shown in Fig. 1 increases the resistance of the concrete as against the linear distribution assumed by the "classic" theory. The "plastic" distribution of the concrete stresses has been used often as the explanation of the excess of the computed compressive strength over the crushing strength of the concrete.

The important feature of Professor Saliger's theory is the demonstration of the possibility of *higher steel stresses than the yield point* under certain conditions, without assuming any modular ratio. The distribution of the steel-strain is not uniform; within a crack and in its neighborhood it is much higher than between two cracks. The average strain is the essential cause of the deformation of the steel, a local peak of the strain (for instance $\epsilon_e = 50$ per mille) is without great influence on the deformation. Therefore it is possible that the steel develops a stress far above the yield point within a crack (according to its stress-strain diagram) whereas between two cracks the concrete is working in tension.

I cannot give here further theoretical details from Professor Saliger's paper which is rather extensive, this short extract however could not be avoided to make this discussion comprehensible.

The characteristic value on which the resistance of a reinforced concrete beam depends is as follows:

We denote

σ_y = yield point of steel

$\mu = \frac{A_s}{bd}$ = percentage of reinforcement

$\beta_y = \frac{\sigma_y}{\sigma_p}$ $\left(= \frac{\text{yield point of steel}}{\text{crushing strength of concrete}}\right)$

The product $\beta_y \mu$ called "strength of the reinforcement" (Bewehrungsstärke) is the characteristic value which is essential for any case under consideration.

Before applying Professor Saliger's theory to the above mentioned tests it is necessary to make some remarks on them. From the point

of view of the maximum bending moment it seems to me essential to separate the two types of loading which were used. The chief difference between them is the fact that by "simple loading" the maximum bending moment due to the superimposed load is constant over the length of $\frac{l}{2}$ and the total bending moment (as the sum of self weight and superimposed load) varies very slowly, whereas in the "continuous loading" (as applied in the described tests) the maximum bending moment arises only at one section and decreases rapidly on both sides of this section (see Fig. 1, p. 4 JOURNAL A.C.I. Sept.-Oct. 1936). The two groups of tests must be considered separately, because the conditions in the case of "simple loading" are more unfavorable for the resistance of the slab though the results are more uniform. In other words, the maximum bending moment under "continuous loading" will probably be higher for a certain beam than under "simple loading," but in a number of tests the deviation from the mean will be greater for "continuous loading." For these reasons we consider only the tests with "simple loading."

Furthermore the tests of slabs reinforced with welded wire fabric are not directly comparable with the other tests, because the typical stress-strain curve is quite different from the corresponding curves for the steel used in the other tests as well as the signification of the "yield point" given in Table 1 (= 80 per cent of the ultimate stress).

In this way we exclude the following tests from further investigation:

a.) Nos. 13-18, with regard to the "continuous loading"
b.) Nos. 4, 20, 30 with regard to the different character of reinforcement
c.) No. 31a with regard to the "concentrated load at the center" (similar to a).

The remaining 19 tests are recalculated in Table A.

Columns 2, 3, 5, 6, 10 are repeated from the original report in order to facilitate the synopsis. Column 8 contains the characteristic values $\beta_y \mu$. The values jd in column 9 are calculated from the values in Table 2, (p. 6) of the report.

Columns 11 and 12 contain the coefficients obtained from the results of the Lehigh experiments which we wish to compare with the coefficients for ultimate strength given by Professor Saliger's theory. The figures in columns 11, 12 are represented in Fig. 2, 3. In Fig. 2 a curve has been drawn in full lines to fit the plotted points as well as possible. The deviation of the single points from the curve do not exceed the usual amounts experienced in experiments with reinforced concrete.

TABLE A

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Test No.	σ_p	σ_y	$\beta_y = \frac{\sigma_y}{\sigma_p}$	d	A_s	μ	$\beta_y \cdot \mu$	jd	M	$\frac{M}{bd^2\sigma_p}$	$\frac{M}{bd^2\sigma_y}$	$\alpha = \frac{\sigma_s}{\sigma_y}$	ϵ_s	x ($k = 0.85$)	$\xi = \frac{x}{d}$	z	ϵ_c
	p.s.i.	p.s.i.		in.	sq. in.	%		jd.	lb. in.				‰	in.		in.	‰
2	2786	47000	16.86	3.12	1.2	1.130	0.1905	2.72	172600	0.1870	0.01110	1.127	17	0.788	0.253	2.78	5.75
3	2700	46000	17.05	3.06	1.2	1.153	.1965	2.66	165300	.1930	.01132	1.125	17	.796	.260	2.72	5.97
5	2690	56600	21.00	3.08	0.75	0.716	.1504	2.77	153800	.1770	.00843	1.307	28	.715	.232	2.78	8.46
6	2784	47500	17.06	3.00	0.77	0.755	.1289	2.67	143300	.1683	.00986	1.470	40	.667	.222	2.72	11.40
7	2640	50000	18.95	3.12	0.80	0.754	.1430	2.78	155300	.1777	.00936	1.400	32	.732	.228	2.81	9.42
9	2890	46000	15.92	3.10	0.35	0.332	.0528	2.84	72500	.0768	.00484	1.595	52	.306	.099	2.97	5.63
10	2795	55000	19.66	3.40	0.35	0.303	.0596	3.10	88700	.0805	.00410	1.487	40	.342	.101	3.25	4.48
11	2600	75000	28.82	3.06	0.35	0.336	.0959	2.79	104500	.1274	.00442	1.430	37	.499	.163	2.85	7.20
12	3390	95500	28.20	3.00	0.35	0.343	.0967	2.73	120000	.1156	.00410	1.318	25	.499	.150	2.81	4.39
21	3100	53100	17.10	3.00	1.43	1.402	.2398	2.61	225400	.2378	.01390	1.135	18	.965	.322	2.59	8.55
22	3140	48500	15.45	3.10	1.40	1.328	.2050	2.70	206700	.2012	.01303	1.130	18	.842	.272	2.74	6.72
23	3085	47100	15.30	3.05	1.55	1.490	.2278	2.66	209400	.2135	.01395	1.091	12	.883	.289	2.68	4.88
24	3110	50300	16.20	2.94	1.32	1.320	.2138	2.56	198900	.2172	.01342	1.173	20	.864	.284	2.57	7.94
25	3085	47000	15.22	3.05	1.20	1.153	.1755	2.66	180400	.1840	.01210	1.200	21	.762	.233	2.74	6.38
26	3000	50000	16.67	3.00	1.00	0.980	.1655	2.64	181300	.1975	.01185	1.374	30	.792	.264	2.66	10.75
28	3200	38950	12.17	2.75	2.00	2.140	.2607	2.34	205750	.2502	.02058	1.127	17	.950	.346	2.34	9.00
29	5260	93000	17.70	2.94	0.77	0.770	.1362	2.62	219300	.1420	.00810	1.170	22	.552	.188	2.70	5.07
31	2880	47300	16.62	2.94	1.60	1.600	.2658	2.50	218000	.2550	.01550	1.134	18	1.048	.356	2.49	9.95
32	5750	50000	8.70	3.12	0.77	0.725	.0631	2.78	147500	.0772	.00888	1.380	30	0.320	.102	2.98	3.43

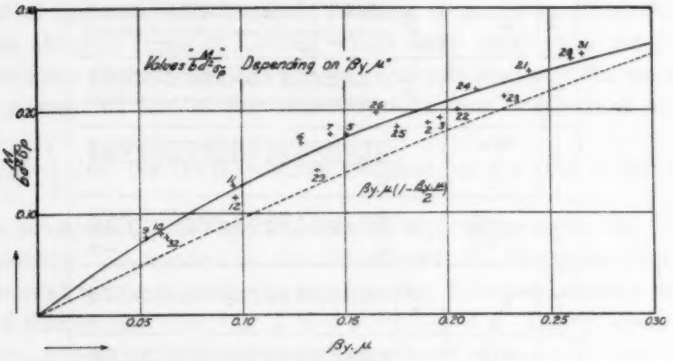


FIG. 2

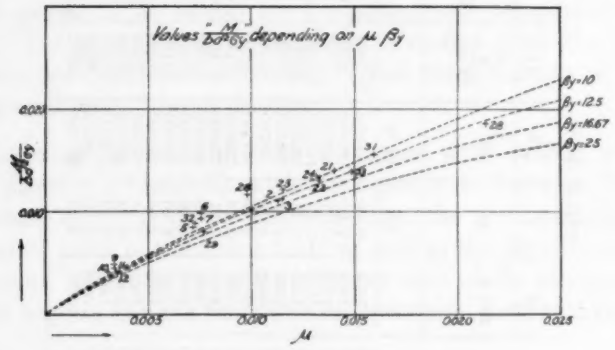


FIG. 3

The dotted lines in the figures give the corresponding values from Professor Saliger's paper. The dotted line in Fig. 2 represents, with a good approximation, the relation between $\beta_y\mu$ and the bending moment at failure; it is based on the assumption that the steel is effective only to its yield point. The excess is characterised by the figures in column 13, shown in Fig. 4, in which the corresponding curve of Professor Saliger is also shown. The resistance of a reinforced concrete section is the result of the combination of the dotted lines in Fig. 2 and 4; the values based on the yield point of the steel are to be multiplied by α which can be calculated if the strengths of the materials and the stress-strain diagrams are known.

In Fig. 3 there are 4 dotted lines corresponding to different values of β_y in Professor Saliger's theory. They differ little in the range of μ

represented in the Fig. ($\mu < 0.025$) and cannot be separated in the Lehigh tests proving the small influence of the strength of the concrete in the tested cases. For increasing values of μ the curves differ more and more and the failure depends more on the concrete strength than on the steel. The coincidence of the plotted points with the dotted lines is very satisfactory. To give the method of calculating the dotted curves in Fig. 3, 4 would require excessive space.‡

It may be of further interest to calculate with a rough approximation the probable strains in the concrete before failure. The strain of the reinforcement according to the excess of the yield point (α) can be seen from Fig. 10 (p. 15) of the report following the three typical curves in which the yield point from Table 2 is assumed to correspond to $\alpha = 1.00$. These figures are given in column 14. In column 15 the position of the neutral axis is calculated according to Fig. 1 with

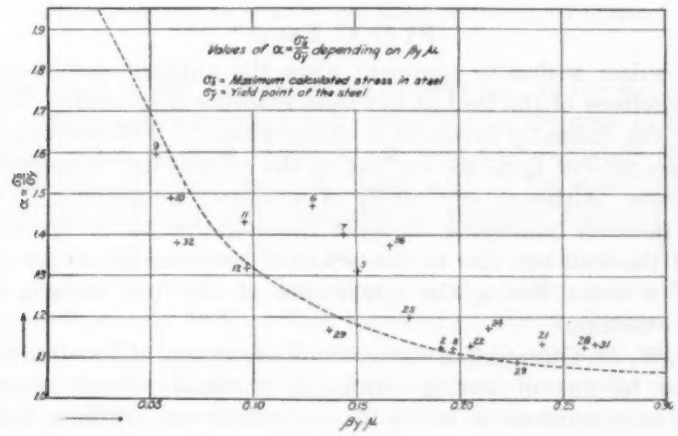


FIG. 4

$$k = 0.85,\ y = \frac{kx}{2} \qquad \text{from} \qquad x = \frac{M}{jd \,.\, b \,.\, 0.85 \,.\, \sigma_p}$$

From x we obtain $z = d - \frac{kx}{2}$ (column 16). The comparison of these figures with jd in column 9 shows very little difference (max. = 4.8 per cent). From the equation

$$\xi = \frac{x}{d} = \frac{\epsilon_c}{\epsilon_c + \epsilon_s}, \qquad \epsilon_c = \frac{\xi \,.\, \epsilon_s}{1 - \xi}$$

‡For further details see *Concrete and Constructional Engineering*, London, Jan.-Feb. 1937 where the new method is tried out also on other tests. (Dr. K. Hajnal-Konyi, "The Modular Ratio: A New Method of Design Omitting m").

we can now calculate ϵ_c (column 18). It varies between 3.43 per mille and 11.40 per mille, only 2 values exceeding 10, in which cases $k > 0.85$ is probable. With $k = 0.95$ the greatest value of ϵ_c (slab No. 6) would be 9.9 per mille. The figures ϵ_c correspond well to the figures observed by Professor Saliger, which coincidence seems to be a further confirmation of his theory.

It is necessary to draw attention to the fact that the values $\beta_y\mu$ of the tests of the Lehigh University vary between 0.0528 and 0.2607 only, whereas the range investigated by Professor Saliger is much wider. Before further generalization the new method should be applied on other results of American tests.

The new method, avoiding any modular ratio, offers designers the opportunity of designing reinforced concrete with a better approximation to the real conditions than the usual method based on a hypothetical modular ratio.

BY G. C. ERNST*

The writer wishes to comment upon the authors' statement that "the exactness of the load at incipient cracking is somewhat doubtful due to the difficulty involved in observation." This annoying fact has been present from the earliest to the present day reinforced concrete tests. Although admittedly of secondary importance, it would be particularly gratifying to most investigators to be certain not only of the load but also of the action of the concrete on the tensile side of a beam during the application of the load causing initial tensile cracking.

Prof. W. M. Dunagan, of Iowa State College, and the writer recently had the fortune of testing reinforced portland cement mortar in tension in a situation in which it was possible to determine the load causing tensile cracking before the crack itself became visible to the eye. The tensile cracks during these tests formed within the 4-in. gage lengths of several mirror extensometers attached to the surface of the mortar. The beginning of tensile failure in the mortar was noted by major fluctuations in the mirror rotations at loads from about 25 to 40 per cent less than the loads at which the cracks were first visible to the eye. The surface of the mortar was white-washed and visual inspection of the surface was made after each set of mirror readings.

The steel strain was measured over a 25-in. gage length completely spanning the mortar upon which the mirror extensometers were set (see Fig. 1). The measured steel strain at first major mirror fluctua-

*University of Maryland.

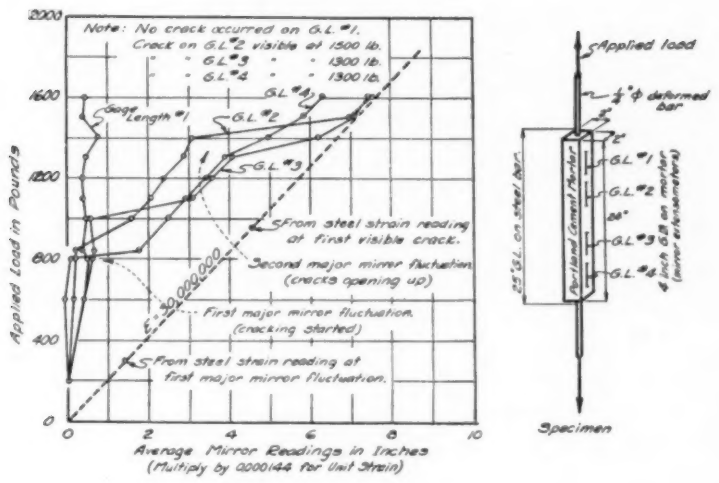


FIG. 1—MORTAR STRAIN VS. LOAD APPLIED TO STEEL

tion was about 0.0002, corresponding to 6000 p.s.i. in the steel as well as the approximate maximum unit elongation for concrete in tension. The steel strain at first visible crack was approximately 0.00065, corresponding to a unit stress of 19,500 p.s.i. in the steel and far in excess of the maximum unit elongation for concrete in tension.

An additional feature was that the *surface* unit strains on the mortar at the start of cracking, as determined by the mirror extensometers, were approximately 50% of that recorded on the steel over the 25" gage length. The mortar in direct contact with the steel, therefore, apparently received about twice as much deformation as the mortar at the surface of the specimen. The thickness of the mortar in this case was one inch. Fig. 1 is submitted for the purpose of showing the type of specimen and the action of the mirror extensometers during the tests. It is interesting to note that the elongation over a 4-in. gage length, after complete cracking, is remarkably close to that of steel alone. It is also evident that, in general, the opening of a crack does not occur suddenly but is gradual, the concrete elongating rather uniformly as long as it can be held together as a unit by the steel. The final points correspond to a steel stress of 32,640 p.s.i. The cracks at this load were still hair-line in appearance and closed upon removal of the load.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . SEPTEMBER-OCTOBER 1936

Metallic Aggregate in Concrete Floors*

By Edward W. Scripture†
MEMBER AMERICAN CONCRETE INSTITUTE

For many years metallic aggregates have been used to impart wear resistance to concrete floors but comparatively few data on this subject have been published. In a booklet issued by the Portland Cement Association it is stated that "Metallic aggregates have been used successfully in the wearing surface of concrete floors. In selecting this material particular attention should be directed to proper grading, freedom from oil, grease, non-ferrous metals and other harmful impurities. It should not be repellent." Although many floors laid with metallic aggregate 15 to 20 years ago are still in excellent condition, a gradual improvement in the aggregates produced has been going on during that time.

Probably the first metallic hardener was simply ground iron to which little attention had been paid with respect to purity and grading. The first step toward improvement lay in the manufacture of a properly graded material so that it would be possible to produce a dense smooth finish without excessive crazing or cracking. It was found that the presence of non-ferrous particles in the iron led to formation of gas, by reaction with the alkali of the cement, producing blisters. Another forward step was the elimination of these non-ferrous particles. Again it was found that oil or grease on the surfaces of the iron particles impaired the bonding of the aggregate by cement so that treatment of the iron to remove oil and grease was started with the result that for a number of years now metallic hardeners of a so called water absorbent instead of a water-repellent nature are available. By grinding into the metallic aggregate a reactive pozzuolanic material strength and resistance to corrosion were improved. One limitation of metallic

*Received by the Institute Secretary, April 29, 1936.
†Director, Master Builders Research Laboratories, Cleveland, Ohio.

aggregates is lack of workability which requires the use of more water and makes application somewhat difficult. This can be overcome by the use of synthetic material in such form that a reduction in water is permitted which improves workability and extends the range of time during which the metallic shake can successfully be applied to the floor.

Simultaneously improvements in the laying of concrete floors were being developed. The rich 1:2 topping mortar was replaced by leaner mixes of graded aggregate which had greater strength and less shrinkage. Better methods of construction permitted the use of mortars with less water. Development of suitable floating machines gave better compacting of the topping and allowed further reductions in water content.

The advances in manufacture of metallic aggregates and in methods of floor construction have been combined to produce floors which should prove far superior to most of the older installations. Briefly, what is usually considered good practice in laying a floor topping with metallic hardeners today is as follows: The hardened base is roughened and cleaned. A bonding coat of cement grout with or without iron is broomed into the surface and this is followed by the placing of the topping mortar. This should be made up of cement, sand from $\frac{1}{8}$ in. down, and aggregate from $\frac{3}{8}$ in.-$\frac{1}{8}$ in. in proportions from 1:1$\frac{1}{2}$:1$\frac{1}{2}$ to 1:1:2 with not more than 4$\frac{1}{2}$ to 5 gal. of water per sack. This is shovelled and raked into place, tamped and screeded. It is then floated with a mechanical float. At the proper time, that is, when no free water appears on the surface but before the topping has taken an initial set, the metallic aggregate mixed with cement (called a shake) is dusted evenly over the surface and floated in with the mechanical float. The topping is then trowelled at the proper time and cured.

Much work has already been done to distinguish between the results obtainable with metallic aggregates of different grades and qualities. It is well known that proper grading, freedom from oil and nonferrous particles are essential to satisfactory results. It has also been shown that corrosion resistance can be increased by pozzuolanic materials if these do not require additional water, and that improved workability and reduced water will give a better floor more easily. On the other hand, although very satisfactory results have been secured by the procedure outlined above, data seem to be lacking on some important points. Among these are, the optimum proportion of cement to be used with the metallic aggregate in the shake, the effect of different mortar compositions on the properties of the finish, the best method of laying the topping, and the most effective quantity of metallic aggregate to use. Consequently, in the program described

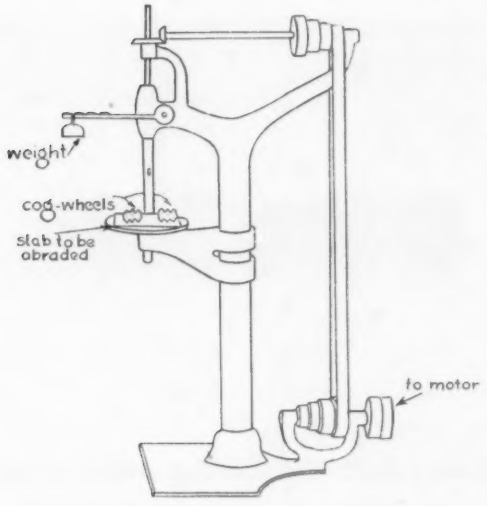


FIG. 1—ABRASION APPARATUS ADAPTED FROM A DRILL PRESS

here the merits of different metallic aggregates were not investigated but the other less well known questions were studied using a single brand of metallic aggregate, known to give good results and complying with all the requirements of a superior material as mentioned above. Similarly, the other materials used, cement and aggregates, were a standard brand of portland cement, a good concrete sand and a well graded siliceous aggregate from $\frac{3}{8}$ in. down and no attempt was made to determine the effects of variations in these. Obviously the conclusions from the results secured are only applicable to materials of the same, or at least similar, nature. Most important in a floor topping are resistance to abrasion and to impact. Apparatus to measure these properties was devised.

EXPERIMENTAL PROCEDURE

To measure abrasion a drill press equipped with a cross bar on which were mounted twenty-six cog wheels over a width of 4 in. (2 in. on each side of a 2 in. center without cogs) was used (Fig. 1). The surface to be tested was placed, in the form of a slab, under this cross bar; the drill press caused the cog wheels to run over the surface at a rate of 180 r. p. m. A fixed load was applied on the handle of the drill press and the operation was continued for a definite time. Measurement of the surface of the slab with a bridge-micrometer before and after abrasion gave the depth of wear (Fig. 2). The bridge extended

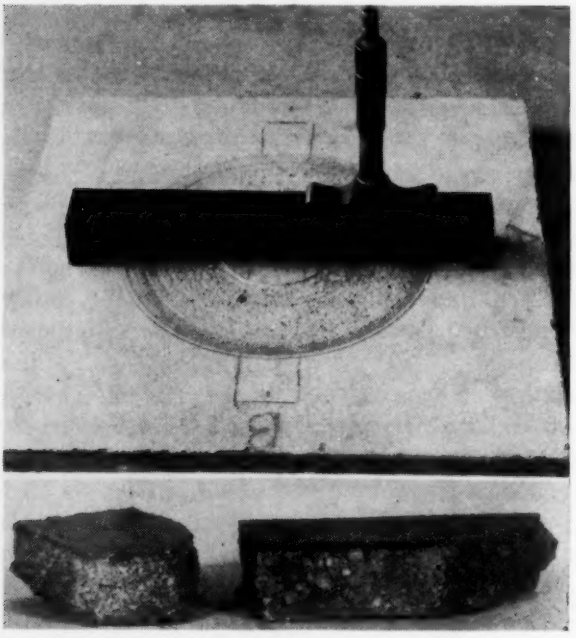


FIG. 2—BRIDGE MICROMETER USED IN MEASURING ABRASION OF TEST SLABS

FIG. 3—CROSS SECTION OF TWO SLABS—LEFT, 30-LB. METALLIC SHAKE—RIGHT, A 120-LB. SHAKE

across the whole diameter (6 in.) of the circular track. Measurements were made of two diameters at right angles to each other. Depth measurements through the holes in the bridge were made at 25 points but since 11 of them fell on the unabraded center, 14 points made up the measurements of actual wear for each diameter. Each abrasion figure in the following tables, therefore, represents the average of 28 measurements on each of two separate slabs. For all the results recorded in this work the average deviation from the mean of the two slabs was 0.0004 in. (eliminating the three poorest checks this deviation was 0.0002 in.). This test is rather more severe than abrasion encountered in most service since the points of the cog-wheels, although free to roll over the surface, tend to cut into it whereas ordinary traffic, including trucks with iron wheels, does not. It gives, however, comparative results which are very consistent and has the great advantage of producing measurable differences in wear in a short time.

The impact apparatus consisted of a frame which permitted the dropping of a half-pound steel ball from successively increasing heights onto the slab to be tested, resting on a felt base. This is similar in principle to apparatus used in testing china. With each slab the point was noted at which any cracking in the top surface first appeared and the point at which the slab actually broke. This test is intended to determine the resistance of the metallic surface itself and the floor topping as a whole to break down under impact such as is produced by the pounding of heavy loads or the dropping of heavy objects.

The procedure followed in making up the specimens for test was in general as follows:

The desired mortar was measured out by volume, mixed in a concrete mixer and placed in an iron frame $\frac{7}{8}$ in. high resting on waterproof paper. The mortar was shoveled into place and raked out level. The frames used were, one 2 ft. x 2 ft. for the specimens where only hand floating was required, and the other 4 ft. x 4 ft. where machine floating was used. The slabs were compacted and finished as called for in the program of the investigation. They were allowed to harden overnight and then cut into 12 in. squares with a carborundum wheel. These squares were cured in water for one week. At the end of this time two of the slabs were removed from the water and immediately broken in the impact apparatus. The remaining two were allowed to dry out in the laboratory for another week and then tested in the abrasion apparatus.

RESULTS

The first series of tests was designed to determine the relative merits of different mortar mixes with and without metallic aggregate. These slabs were all placed and finished by hand, hence are not strictly comparable to the later slabs of similar mixes placed with a special tamp or machine floated. The mixes used were a 1:2 mortar of standard portland cement and a good concrete sand, a 1:3 mortar with the graded silica aggregate and a similar 1:3 mortar with 25 per cent of a metallic hardener used integrally. Results of the tests on these slabs are given in Table 1.

TABLE 1

Mortar Mix	Metallic Aggregate Lb. per 100 sq. ft.	Abrasion Depth of Wear in Inches	Height of Drop (In Ft.) to Cause Impact Failure		
			Surface Cracked	Broke	Average
1:2	None	.018	3¾	4¼	4
1:3	None	.014	3¾	4¼	4
1:3	60	.012	4	4½	4¼
1:3 with hardener used integrally	60	.008	5¼	5¾	5½

It is evident that the better designed mix, that is the 1:3 mix with aggregate graded up to ⅜ in. shows a markedly greater resistance to abrasion than does the richer 1:2 mix. Similarly the metallic aggregate on the surface provides a further improvement in resistance to abrasion. These advantages might well be expected. The better graded mix, although it probably gains little in compressive strength, is denser and would be expected to resist abrasion better than the richer mix. Again the rich mix has a larger proportion of cement in the surface which wears more rapidly than the aggregate. The action of the metallic aggregate is somewhat different in that, due to the ductility of iron, it tends to be flattened out and compacted with wear rather than to break down as do equally hard or harder but more brittle aggregates. The integral use of a metallic hardener has still further reduced abrasive wear.

With respect to impact there is little to choose among the first three mixes. Aside from any tendency for the metallic surface to act independently the resistance in this test probably reflects compressive or flexural strength and this would be approximately equal for these three mixes. Evidently 60 lb. of metallic aggregate per 100 sq. ft. is not sufficient to give any marked increase in the strength of a ⅞ in. topping. That the bonding of the metallic aggregate with the mortar is good may be inferred from the absence of any surface cracking at impacts lower than those which crack the slabs without metallic aggregate and from the observation that when cracks do appear they are normal and the metallic surface shows no tendency to chip. The mix which contains the metallic hardener* integrally shows an increased resistance to impact due to greater strength.

The second series of experiments was designed to determine the best proportions of cement to be used with the metallic aggregate. The same mortar, 1:3 with graded silica aggregate, was used throughout. The slabs were placed, machine floated and finished by trowelling in

TABLE 2

Lb. per 100 sq. ft. Metallic Aggregate	Cement	Depth of Abrasion in Inches	Height of Drop (In Ft.) to Cause Impact Failure		
			Surface Cracked	Broke	Average
30	22½	.003	4.5	6.3	5.4
30	15	.002	6.1	6.8	6.5
30	7½	.004	6.4	7.4	6.9
60	45	.003	6.5	7.0	6.8
60	30	.003	6.0	6.4	6.2
60	15	.003	6.0	6.5	6.3

*The metallic hardener used integrally is not the same material as that used on the surface. The later is primarily a well-graded metallic aggregate free from oil, grease and non-ferrous particles containing a synthetic pozzuolana which permits reduction of water ratio. The former, although based on a similar metallic aggregate, also contains considerable proportions of other ingredients.

the same manner in all cases. The only variable is the proportion of cement to metallic aggregate in the shake, in dust coats of two different thicknesses. Results of tests are given in Table 2.

With the lighter shake the 4:2 proportion of metallic aggregate to cement shows markedly higher abrasion resistance than either the higher or lower proportions. With the heavier shake (60 lb. per 100 sq. ft.) the differences are small but the 4:2 and 4:1 proportions are slightly better than the 4:3 mix. Differences in impact resistance are all so small that they do not seem to have any significance except that the lighter shake cracks at a very low impact with the rich 4:3 shake. It is recognized that excessively rich mixes are not desirable because of high shrinkage and the danger of crazing and checking. These tests show that the leaner mixes are at least as good and generally better in resistance to abrasion and impact than the commonly used rich shake. The limit in leanness is, therefore, determined not by strength but by workability. It was noticeable in preparing the test slabs of this series that the 4:1 shake was difficult to finish properly but the 4:2 mix was satisfactorily workable. The latter would seem to be the most desirable proportion of metallic aggregate to cement.

It is of interest to compare the two comparable slabs of the first and second series. It will be noted that in resistance to both abrasion and impact the machine floated specimens are far superior to those placed and finished entirely by hand.

Comparison of the lighter (30 lb. per 100 sq. ft.) and the heavier (60 lb. per 100 sq. ft.) shakes indicates that the amount of metallic aggregate within these limits has little or no effect on resistance to impact. It is likewise true that the amount of the shake does not materially affect the rate of abrasion. Of course, this only holds as long as the abrasion does not wear through the thickness of the metallic layer; beyond that point the rate of abrasion will change. Consequently the heavier shake, although wearing at the same rate will last longer.

A third series of tests was made to investigate a new type of tamp. This tamp consisted of a series of 25 parallel steel bars $\frac{3}{16}$ in. thick, spaced $\frac{1}{4}$ in. apart suitably mounted on a handle. It was used to compact the topping after it was raked into place approximately level. After screeding off, the topping was again tamped, the metallic shake dusted on and tamped in. The heavier shake was applied in two portions. After the final tamping the slab was finished either by hand trowelling alone or by machine floating followed by hand trowelling. All the shakes used were mixed in the proportions, 4 parts metallic

aggregate to 2 parts cement by weight. Results of the third series of tests are given in Table 3.

TABLE 3

Lb. per 100 sq. ft. Metallic Aggregate	Method of Finishing	Depth of Abrasion in Inches	Height of Drop (In Ft.) to Cause Impact Failure		
			Surface Cracked	Broke	Average
60	Tamp Only	.006	5.7	6.3	6.0
60	Machine Only	.003	6.0	6.4	6.2
60	Tamp and Machine	.003	5.9	6.9	6.4
150	Tamp Only	.004	6.2	7.0	6.6
150	Machine Only	.002	6.9	7.6	7.3
150	Tamp and Machine	.002	6.0	6.8	6.4

This remarkably consistent series of results shows that machine floating produces better resistance than tamping alone, but that tamping and machine floating are superior to either alone.

Comparing the heavier with the lighter shake there appears to be some advantage for the latter even in rate of abrasion; obviously there would be a distinct increase in life. While in the previous series there was not much difference in resistance to impact between a 30 lb. and a 60-lb. shake, there seems to be a definite increase between a 60 lb. and 150-lb. metallic shake. Evidently in the heavier shake, a sufficient metallic layer has been formed to add measurably to the strength of the topping.

It might seem that preference should be given to machine floating over the tamp method where only one of these can be used, but this is not entirely justified. With the lighter shake it is probably true; not with the heavier. While it was possible to prepare test specimens of this type in the laboratory, it would not be feasible in practice to introduce 150 lb. of metallic aggregate per 100 sq. ft. into the topping by machine floating; it is entirely practicable to do so with this special kind of tamp.

Believing from the previous experiments that there might be some advantage in the application of fairly large quantities of metallic aggregate with the special tamp, another series of tests was undertaken to compare the two common types of topping mortar which might be

TABLE 4

Mortar	Lb. per 100 sq. ft. Metallic Aggregate	Depth of Abrasion in Inches	Height of Drop (In Ft.) to Cause Impact Failure		
			Surface Cracked	Broke	Average
1:2	60	.004	6.1	6.9	6.5
1:3	60	.006	5.7	6.3	6.0
1:2	150	.004	7.0	7.9	7.5
1:3	150	.004	6.2	7.0	6.6

Water used—3.6 gals. per sack of cement for1:3 mortar.
Very slightly lower for 1:2 mortar.

used for this purpose. A 1:2 cement-sand mortar was compared with a 1:3 cement graded silica aggregate mortar with two different weights of metallic aggregate. The slabs were all laid and compacted with the tamp and finished as previously described. Results of tests are given in Table 4.

Since wear is dependent on the qualities of the surface it is perhaps not suprising that the 1:2 and 1:3 mortars show little difference in resistance to abrasion. It is more surprising that the 1:2 mortars give slightly greater resistance to impact than the 1:3. The reason is probably that the 1:2 mortar can be compacted with the tamp at slightly lower water content than the 1:3. While there does not appear, from these tests, to be a great deal of choice, the 1:3 mortar is probab y preferable for two reasons; greater economy and lower shrinkage.

After a consideration of the variables involved in laying a topping and finding an advantage for metallic shakes of greater thickness than has been usual, it is desirable to learn just how great a thickness is beneficial. Since the heavier shakes can only be applied in practice by means of the special tamp a series of slabs was prepared by the tamping method previously described with the 1:3 graded silica mortar and various weights of metallic aggregate. Results are given in Table 5.

TABLE 5

Lb. per 100 sq. ft. Metallic Aggregate	Depth of Abrasion in Inches	Height of Drop (In Ft.) to Cause Impact Failure		
		Surface Cracked	Broke	Average
30	.006	5.0	6.4	5.7
60	.006	5.5	6.5	5.0
120	.003	6.8	7.8	7.3
150	.004	6.2	7.0	6.6

This very consistent series of slabs shows that resistance to abrasion, to cracking and to breaking on impact increase with the amount of metallic aggregate applied up to 120 lb. per 100 sq. ft. Above this point resistance to these forces begins to fall off, probably due to the limitations of the method of application.

It should be noted that, although there is some gain in rate of abrasion with the heavier shakes the advantage in longer life should be even more marked. For example, while the rate of wear is only half as great with the 120-lb. shake as with the 30-lb. shake the depth to be worn through is four times as great. It is also worthy of mention that the metallic layer becomes an integral part of the topping showing no tendency to chip off under impact at loads up to the breaking point of the slab.

It was noted in the first series that some very good results were secured with the integral use of a metallic hardener. In the later series good resistance was imparted by the use of a large quantity of metallic aggregate on the surface. A comparison of the two methods would, therefore, be enlightening. Two slabs were made up with the integral hardener and metallic aggregate shake, finished by machine floating and troweling. Corresponding slabs were made up without the integral hardener but using the tamp instead of the machine float. The 1:3 mortar was used in all cases. Results of these tests are given in Table 6.

TABLE 6

Integral Metallic or Tamp	Lb. per 100 sq. ft. Metallic Aggregate	Depth of Abrasion in Inches	Height of Drop (In Ft.) to Cause Impact Failure		
			Surface Cracked	Broke	Average
Integral	60	.003	6.2	6.9	6.6
Tamp	60	.006	5.5	6.5	6.0
Integral	150	.003	5.6	6.4	6.0
Tamp	150	.004	6.2	7.0	6.6

It is obvious that the abrasion resistance is better with the metallic hardener used integrally. On the other hand the impact resistance, while better with the integral metallic in the case of the lighter shake, is not as good with the heavier shake. This is presumably due to the difficulty of application of the thick metallic layer by machine. It should also be noted that the 120-lb. shake (Table 5) applied by the tamping method shows abrasion resistance equal to the slabs in which metallic was used integrally and superior resistance to impact.

A few general observations should be made respecting the practical application of these floor finishes. The heavier applications of metallic aggregate or the integral use of a metallic hardener will add substantially to the cost of the floor. Observation of a large number of installations leads to the conclusion that, for ordinary and even fairly severe conditions of service the shake of 30 lb. to 60 lb. per 100 sq. ft. is adequate. For some exceptionally severe conditions, however, there would be distinct advantages in the more costly shakes up to 120 lb. per 100 sq. ft. Although laboratory tests indicate that good results can be produced by machine floating, it does not seem practicable to apply these large quantities of metallic aggregate in the field without the use of special tamping.

CONCLUSIONS

While it is recognized that the number of specimens is limited, the consistency of the results leads to the belief that they present an

accurate picture of the variables investigated. Consequently, tentative conclusions have been drawn as follows:

1. Leaner mortars with aggregate graded up to ⅜ in. are superior to the rich 1:2 mortar.

2. The wearing qualities of a topping can be substantially enhanced by application of a suitable metallic aggregate to the surface.

3. The wearing qualities may also be improved by the integral use of a metallic hardener.

4. Best results are secured with metallic aggregate and cement in about the proportions of 2:1 (by weight) for the shake.

5. For metallic shakes up to 60 lb. per 100 sq. ft. machine floating is superior to the special tamping method investigated, but use of both tamp and machine float is superior to either alone.

6. For shakes greater than 60 lb. per 100 sq. ft. the only practical method of application is by means of the special tamp.

7. The highest resistance to abrasion and impact is secured with 120 lb. of metallic aggregate per 100 sq. ft.

8. Where the tamping method is used there is not much difference between the 1:2 and 1:3 mortars as far as shown by the tests applied, but the latter is probably preferable.

9. There is not much difference in rate of wear between the best results obtainable with metallic shakes using the tamping method and using metallic hardener integrally. The tamping method should, however, give longer life due to the greater thickness of aggregate which can be applied.

10. Metallic aggregate in weights from 30 lb. to 60 lb. applied by machine floating should give sufficient wear resistance for all ordinary service conditions; for very severe conditions a distinct advantage can be realized with the use of 120 lb. of metallic aggregate per 100 sq. ft.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for January-February 1937. Discussion should reach the Institute Secretary by December 1, 1936.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY, 1937

Discussion of a paper by Edward W. Scripture:

Metallic Aggregate in Concrete Floors*

E. ROELOFSEN†

The type of floors having an upper layer of portland-cement and metallic aggregate is far different from that made in Europe with the aid of metallic aggregate. Mr. Scripture discusses a floor, having a layer of mortar (1 part cement—three parts graduated silica aggregate) in which a shake consisting of two parts metallic aggregate and one part cement is fixed. The abrasion-test teaches us something about the quality of the surface. The influence of the proportions of cement and metallic aggregate is studied in detail but from our own investigations it also seems necessary to trace the influence of the granulometric composition of the metallic hardener. It appeared to us that the abrasion is influenced very strongly by this granulometric composition and that mixtures with much fine material show greater abrasion. This influence is far greater than the influence of the quantity of cement used in the mixture.

The test with the impact apparatus gives an idea of the quality of the upper layer resistance to the shocks of falling objects. As Mr. Scripture says, the qualities of the 1 - 3 mortar appear.

When constructing an industrial floor it is of most importance that it be not destroyed by falling objects. It is necessary in our opinion not only to examine the influence of the quantity of cement in the mortar and the granulometric composition, but moreover to trace the influence of the kind of aggregate. On examination we found that mortars with a tough material have a much greater resistance to falling objects than mortars with a brittle material.

*JOURNAL Amer. Concrete Inst., Sept.-Oct., 1936, p. 17.
†Chemist of the laboratory of N. V. Betonfabriek "De Meteoor," De Steeg, Holland.

Moreover rubbing in a small quantity of a mixture of metallic hardener and cement in the upper layer causes irregularity in the quantity of the metallic hardener per unit of surface and consequently a destruction of the floor by the wheels of cars and trucks begins here. Experience teaches us that when destruction begins it enlarges itself quickly.

When constructing industrial floors for heavy duty in Europe we consider the following:

1. The sub-concrete-floor should be of very good quality; therefore a good granulometric composition of the aggregate, and correct water-cement factor.

2. On the floor a thin mortar-layer of 1:2½ or 1:3 sand is affixed —as thin as possible (0.5 cm.), to serve only as a binder between the sub-floor and the upper layer.

3. On this an upper layer (thickness ± 1 cm.), consisting of 1 unit by weight of portland cement and 2.5 units by weight of metallic hardener, is affixed. We call it a *protecting layer*, not a topping. For this mortar layer there are the following requirements: great resistance to abrasion and great resistance to the shocks of falling objects. Accordingly we had to make many tests to attain the highest quality of this 1 cm. upper layer.

From this it appeared that the granulometric composition of the metallic hardener used is of great importance in resisting abrasion and the tensile strength for resistance to falling objects.

The metallic aggregate that has a long-drawn prismatical shape is composed in consequence as follows:

50 per cent ± 2 m.m. x 6 m.m.
25 per cent ± 1 m.m. x 3 m.m.
25 per cent 0 m.m. to 1 m.m.

Results are influenced as mentioned by Mr. Scripture by right water-cement factor, good compression, etc.

In comparing tests of such metallic hardeners used in Europe, and other kinds of hardened concrete floors it always appeared that the abrasion is much less and the resistance against falling objects is much greater; splittering of the surface does not appear.

This is of value where jointless flooring is made. However in several countries of Europe there is an increasing tendency to make floors with flags. By this it is possible to make invisible repairs and replacements, while the machine-made manufacture of the flags guarantees a superior quality, and great density.

Most of these flags are 300 x 300 m.m. (12 x 12 in.); thickness 2 in., 1½ in. or 1 in., including a metallic hardened upper layer of 5 m.m. (0.2 in.), 8 m.m. ($\frac{5}{16}$ in.) or 10 m.m. ($\frac{3}{8}$ in.).

Flags with a thickness of but 10 m.m., consisting of a mortar of cement and metallic hardener are also made; the metallic hardened surface is then delivered as a flag.

We hope to have succeeded in giving you in this small compass an idea of the state of the problem of metallic hardened floors in Europe.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . SEPTEMBER-OCTOBER 1936

Evaluating Fines in Concrete on a Bleeding Test Basis*

By J. C. Sprague†
MEMBER AMERICAN CONCRETE INSTITUTE

INTRODUCTION

This investigation was made to determine the amount of an admixture required to produce the same degree bleeding of mixing water in concrete as that produced by natural fines passing the No. 100 screen. The investigation was conducted in the District Testing Laboratory of the Huntington Engineer District, War Department. The program was designed to develop a reliable method of test for measuring water-gain in concrete when slight changes were made in the proportion of fines in the aggregate. It is believed that the method used in this investigation, hereafter referred to as the "bleeding test," will give a sufficiently reliable measure of comparative workability for variations in fine materials, and that repeated tests will produce consistent results.

Concrete mixtures, in which varying percentages of different types of fine materials are incorporated, are compared with a control mixture on a basis of the bleeding action of mixing water. In these tests, 9 types of fines were used in 19 different combinations.

In general, considering the various materials in these tests, used as fines in lieu of natural fines, those of high normal consistency showed less bleeding action than those of low normal consistency. In a few instances the reverse was true, but in such cases the specific gravities of the materials were usually higher and the materials were not as finely ground.

Current specifications in force in this District and other districts of the Corps of Engineers, U. S. Army, stipulate that a minimum of 2 per cent fine aggregate pass the No. 100 screen. If aggregate contain-

*Received by the Institute Secretary, July 26, 1936.
†Assistant Engineer, U. S. Engineer Office, Huntington, W. Va., in charge of the District Testing Laboratory.

ing such fines is not available, an approved admixture shall be used to meet the deficiency. In anticipation of the above, tests were made to determine what proportion of each of several different commercial admixtures would be required to produce concrete having a percentage of water-gain equal to that produced by natural fines passing the No. 100 screen.

The usual field method of measuring consistency of concrete, i.e., the slump test,[1] was not considered as it was not believed to be sufficiently sensitive to show accurately the changes resulting from such slight variations in the mix as would be required for these tests. The flow test,[2] although more sensitive than the slump test, was rejected for the same general reasons, although the mechanical principle of the flow table was adapted to this investigation. After considerable study, the method described in this report for evaluating fines in concrete on a bleeding test basis was decided upon. This decision was influenced by the assumptions: (1) That segregation in concrete is influenced by gradation; (2) That water-gain in concrete is a direct result of segregation.

SCOPE, DESCRIPTION AND RESULTS OF TESTS

Tests were made on 19 different combinations, one of these being a control mixture containing the specified 2 per cent fines. These combinations included (a) a mixture containing 3 per cent natural fines, (b) a mixture completely lacking in fines, and (c) 16 mixtures containing different percentages of commercial fines used as admixtures.

The actual quantities of materials required for each batch of concrete were calculated on an absolute volume basis, and were then determined in terms of weight for a 1/12 bag mix (0.45 cu. ft. of concrete).

The fine aggregate was separated into its particle sizes of 0-100, 100-50, 50-30, etc. A sufficient quantity of each size was weighed for each batch, so as to secure a synthetic grading which had previously been determined. By following this procedure, all tendency toward initial particle segregation in any one mixture was avoided. The coarse aggregate was divided into "large" and "small" sizes, the large size having 100 per cent passing the 2-in. screen and retained on the 1 in., and the small size having 100 per cent passing the 1 in. and retained on the No. 4 screen. A cement factor of 1.25 and a water-cement ratio of 0.827 were used throughout, as these values represented

[1] A. S. T. M. Tentative Method for Consistency of Portland-Cement Concrete (D138-32T) 1934 Book of A. S. T. M. Tentative Standards, p. 723.

[2] Described in the A. S. T. M. Standard Methods of making compression tests of concrete (C39-33) 1933 Book of A. S. T. M. Standards, Part 2, p. 230.

an average field condition. The ratio of fine to total aggregate were 33⅓ per cent, and the ratio of small and large gravel was 70-30. After the weights of the concreting materials had been calculated, weights of each particle size of the fine aggregate were determined from the following gradation:

Sieve No.	Cumulative % Retained	% Retained on Each Sieve	Weight of Each Particle Size
4	4%	4%	0.741 lbs.
8	13%	9%	1.667 lbs.
16	25%	12%	2.223 lbs.
30	53%	28%	5.188 lbs.
50	90%	37%	6.856 lbs.
100	100%	10%	1.853 lbs.
F. M. =	2.85	100%	18.528 lbs.

This synthetic grading was made to simulate as nearly as possible the average grading of fine aggregate used in concrete in the District. Before these quantities were computed the 2 per cent natural fines passing the 100 mesh screen were deducted from the total, as follows:

$$18.906 \times .02 = 0.378 \text{ lbs. of natural fines.}$$
$$18.906 - 0.378 = 18.528 \text{ lbs. of fine aggregate.}$$

All other materials, used as admixtures in this investigation, were substituted for the natural fines in the control mixture on a basis of weight. In the early part of the tests the substitution was made on a basis of pound for pound. Thus an equal weight of each of the admixtures used were substituted for the weight of fines in the control.

To eliminate the effect of other variables such as temperature changes, differences in humidity, and the personal equation in manipulation, the samples were always made in pairs, one of the samples in every case being the control mixture. In this way there was always a direct comparison between the control and the mix under investigation, though the actual values varied from day to day.

The chemical and physical characteristics of the various materials used in this investigation are given in Table 1. Attention is invited to the great dis-similarity between the specific gravities and normal consistencies of the various materials, as these characteristics are believed to play an important role in the behavior of the materials in these tests.

The tests in this investigation were divided into two series. In the first series equal parts by weight of each material were substituted for the natural fines in the control mix in the manner described above. This was done because it was thought desirable to know just what the effect of each, substituted on an equal weight basis for natural fines, would be on the bleeding action of water in the concrete. The results obtained by using this means of comparison showed a more pronounced

TABLE 1—COMPOSITION OF MATERIALS AND CALCULATED PER CENT OF EACH REQUIRED TO PRODUCE SAME PASTE CONTENT AND WATER-GAIN AS ONE PART OF NATURAL SUB-SIEVE FINES

Material Designation	Physical Characteristics					Chemical Composition							Sp. Gr. of Pastes @ Normal Consist.	Part by Weight to Produce eql. Paste	Solids and Water in the Paste				Part by Weight to Produce Same Bleeding Action as 1 Part of Natural Fines	
				Fineness (Passing)											Weight		Abs. Vol.			
	Specific Gravity	Normal Consistency	Weight per cubic foot	No. 100 Sieve	No. 200 Sieve	Ignition Loss	SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO	SO_3			Sol	H_2O	Sol	H_2O	Calculated	Actual
		%	Lbs.	%	%	%	%	%	%	%	%	%			%	%	%	%		
E	2.65	23.5	90	90	67	—	—	—	—	—	—	—	2.02	1.07	81	19	62	38	1.17	.97
Nat. Fines	2.65	27.5	88	100	54	3	—	—	—	—	—	—	1.95	1.00	78	22	58	42	1.00	1.00
A	2.54	31.0	67	98	93	5	51	20	15	7	1	3	1.86	0.93	76	24	56	44	0.93	0.81
C	2.54	31.5	67	97	93	3	45	22	17	4	1	1	1.85	0.92	75	25	55	45	0.91	1.08
B	2.65	40.0	57	99	85	12	3	2	6	67	2	2	1.80	0.84	71	29	49	51	0.69	0.70
G	2.55	50.0	55	90	80	1	97	1	—	—	—	0	1.68	0.73	67	33	44	56	0.57	0.30
D	2.07	81.0	79	100	98	5	70	11	14	—	—	—	1.40	0.50	55	45	37	63	0.43	0.33
F	2.06	152.0	13	100	95	5	90	Approximately 5%					1.26	0.33	40	60	24	76	0.23	0.11
Port. Cem.	3.28	23.5	94	99	88	1	23	4	5	63	3	2	2.29	1.21	81	19	57	43	0.99	0.93

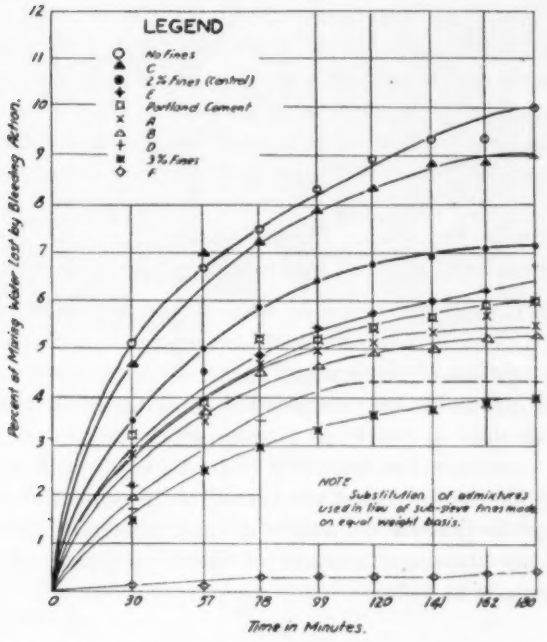


FIG. 1—EFFECT OF DIFFERENT ADMIXTURES AS COMPARED WITH 2 PER CENT SUB-SIEVE FINES IN SAND ON THE BLEEDING OR WATER-GAIN IN CONCRETE

effect on the water gain in concrete than was anticipated, and indicated that the bleeding test was very sensitive to changes in characteristics of the fine materials. The results of these tests are shown in Fig. 1. It will be noticed that the curve for the control mix shows that slightly over 7 per cent of the total mixing water in the concrete had been lost through bleeding action at 3 hours; this curve represents the average of over 100 determinations. Tests were continued for only 3 hours in most cases as initial set took place shortly after that time and the acceleration in water gain had dropped perceptibly, the curves becoming practically horizontal. A few tests were made in which the time was extended to 4 hours but there was very little water gain over the standard time set, i.e., 3 hours. The curve representing the mix which contained no fines shows an increase over the control curve of about 33 per cent. In other words, the water lost in this mix was 10 per cent or nearly 3 per cent more than for the control. Similarly, admixture C showed considerably more bleeding than did the control.

At the other extreme it will be noticed that admixture F showed very little water gain. By referring to Table 1, and noticing the physical characteristics of the materials, it will be noticed that admixture F has a comparatively low specific gravity and a very high normal consistency. In other words, because of its low specific gravity, less of this material by weight is required to produce an absolute volume equal to that of the natural fines and, because of its high normal consistency, an equal weight of it will hold about 5½ times as much water as will the natural fines. The other materials tested show water losses ranging between this material and the control mixture.

In making the second series of tests it was the aim to determine what per cent of each material would be required to produce bleeding action equal to that of the control mix. To save time and material which would otherwise be consumed in making trial batches it was decided to calculate as nearly as possible from the physical characteristics of the materials the quantity of each required in a mixture to produce bleeding equal to that produced by the control mixture. The calculation for the per cent by weight of each material to use to produce concrete having the same amount of bleeding action as the control mixture was made as follows:

$$X = \frac{ab}{cd}$$

Where: X = per cent by weight required of the admixture.
a = specific gravity of natural fines.
b = normal consistency of natural fines.
c = specific gravity of the admixture.
d = normal consistency of the admixture.

Tables 1 and 2 show the values for each material found by this method of calculation.

Checking by Trial. The computations referred to, (the results shown in Tables 1 and 2) were used as the first approximations in determining what part by weight of each material would be required to produce a bleeding action in concrete equal to that produced by the mixture containing 2 per cent natural fines. The final mixes were determined by trial, using the type of apparatus shown in Fig. 2. The apparatus used, referred to hereafter as the "water-gain" table, is based on the same general mechanical principle as that of the flow table.[3] The top of the water-gain table is hinged in the back, and the front is free to move up and down when a shaft, to which cams are attached, is revolved. There are two cams spot welded to the shaft at third way points and these cams, when the shaft is turned, come in contact with strips of steel fastened to the underside of the top, raising the top and allowing it to

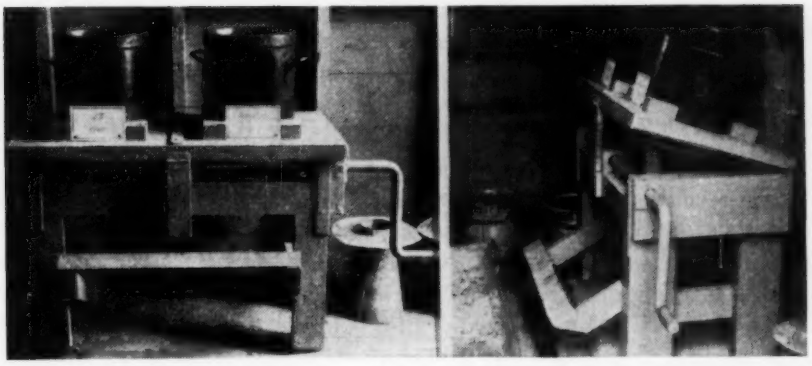


FIG. 2—APPARATUS USED IN BLEEDING TESTS—LEFT, NORMAL POSITION; RIGHT, INCLINED FOR EXTRACTING WATER

drop ¼ in. Two half-cubic foot steel cylindrical measures, containing a control mixture of concrete and a mixture of one of the other materials, are set in grooves in the table top and the agitation caused by the drop of the top accelerates the bleeding action in the mixtures. The table is set upon a concrete foundation to minimize the effect of cushioning action.

The quantity of each material, used as an admixture in the preliminary trial mixtures, is shown as "calculated part by weight" in Table 1. After the ingredients had been weighed they were dry mixed for 3 minutes on water-tight mixing boards. The dry materials were then built into a mound, a hole scooped out in the center and water added to each at the same time. The mixing water was allowed to soak in for one minute and the concretes were then thoroughly mixed for 5 minutes. At the end of the first half of this mixing period, the two operators doing the mixing changed positions. This was done to minimize the personal equation entering into the work. At the end of the 5 minute mixing period the concrete was scooped into the steel containers. The specimens were then weighed and placed on the table. The first photograph in Fig. 2 shows the specimens in their proper position at the beginning of the test. Twenty-five minutes after completion of molding, the samples were agitated 24 times in 24 seconds. At the end of the 24th drop the top of the table was raised to the inclined position as shown in the second photograph of Fig. 2. At the end of another 5 minutes, two operators simultaneously drew off the water which had risen to the top of the concrete. One minute was

[3]G. M. Williams, "Concrete Consistency Measured by the Flow Table," E. N. R., Vol. 84, No. 22, p. 1044 (May 27, 1920).

TABLE 2—PROPORTIONATE PARTS OF THE DIFFERENT MATERIALS REQUIRED TO PRODUCE BLEEDING ACTION EQUAL TO THAT PRODUCED BY 1 PART OF NATURAL FINES

Designation of Material Used as Fines	Description of Material	Part Used By Weight to Replace 1.0 Part of Nat. Fines	Per Cent By Weight of Original Mixing Water Lost Through "Bleeding" (Time in Minutes)								Calculated Parts for Equal "Bleeding" $X = \frac{\text{(Sp. Gr. Sand) (N. C. Sand)}}{\text{(Sp. Gr. Ad.) (N. C. Ad.)}}$ Sand: 2.65 x 0.275 = 0.729
			30	57	78	99	120	141	162	180	
			%	%	%	%	%	%	%	%	
2% Natural Fines	River Run Material Passing the No. 100 Sieve	1.00	3.5	5.0	5.8	6.4	6.7	6.9	7.1	7.2	$\frac{0.729}{(2.65 \times 0.275)} = 1.00$
Portland Cement	Blend of 6 Cements	0.93	3.8	5.1	5.9	6.2	6.5	6.8	6.9	7.1	$\frac{0.729}{(3.28 \times 0.235)} = 0.99$
A	Precipitate Ash	0.81	3.2	5.0	5.8	6.4	6.6	7.0	7.2	7.3	$\frac{0.729}{(2.54 \times 0.31)} = 0.93$
B	———	0.70	3.2	4.6	5.6	6.4	6.8	7.2	7.4	7.4	$\frac{0.729}{(2.65 \times 0.40)} = 0.69$
C	Amorphous Colloidal Clay	1.08	3.5	5.7	5.9	6.6	6.8	7.1	7.3	7.4	$\frac{0.729}{(2.54 \times 0.315)} = 0.91$
D	Colloidal Suspension Paste	0.33	1.7	4.0	4.8	5.8	6.5	7.0	7.3	7.4	$\frac{0.729}{(2.07 \times 0.81)} = 0.43$
E	Pulverized Siliceous Material	0.97	3.3	4.9	5.7	6.5	6.8	7.0	7.2	7.4	$\frac{0.729}{(2.65 \times 0.235)} = 1.17$
F	Diatomaceous Silica	0.11	3.1	4.1	5.0	5.7	6.1	6.3	6.5	7.0	$\frac{0.729}{(2.06 \times 1.52)} = 0.23$
G	Meta-Colloidal Silica	0.30	2.7	4.2	5.1	5.3	5.8	5.9	6.5	7.0	$\frac{0.729}{(2.55 \times 0.50)} = 0.57$

allotted for this operation after which the table top was lowered to its normal position. The water thus removed was weighed and the weight recorded. The same procedure of agitation, bleeding, etc., was followed for the remainder of the test, the agitations recurring at the following time intervals of 57, 78, 99, 120, 141, 162 and 180 minutes from the completion of molding. At the end of 180 minutes each container was immediately weighed and the weight recorded. The original weight minus the final weight gave the approximate weight of the water which was drawn off the concrete. This figure was used as a check against the direct weight of the water extracted, although the latter weight was used since it was taken to the nearest gram and was therefore more accurate.

The water which was drawn from the concrete represented that part of the mixing water which was lost through bleeding action. This loss is expressed in terms of per cent by weight of original total mixing water entering the concrete. Table 2 shows the per cent of bleeding at each time interval. A comparison between the per cent of each material required to produce equal bleeding (Table 2) and the per cent of bleeding in mixtures containing equal amounts of fines (Fig. 1) shows the same trend in each case.

In all cases, except two, the actual weights of materials required to produce equal bleeding action was found to be less than that calculated. In one of the excepted cases (admixture B) the first trial mixture agreed with that calculated; the other excepted case (admixture C) has already been mentioned. In no case was it necessary to make more than three trials before the correct quantity was determined. After this quantity had been determined, the test was repeated a number of times and all results varying from the average more than 10 per cent were discarded.

One thing of interest noticed in making these tests was the rate of water gain at different periods. Fig. 3 shows curves for each material, the points on the curves representing the percentage of total water lost during the test at each increment of time. Admixture D shows a slower gain than any of the other materials. The curves representing "no fines" in the concrete shows the fastest gain up to 30 minutes, after which it follows the curve for the control mixture. Curves for the other materials fall between these two, except for material G which shows a pronounced deceleration from 57 minutes to the end of the test period.

From the standpoint of economy it is evident that any test for evaluating amounts of different types of fines for use in concrete must be very accurate. In a project of any great magnitude, an error of a

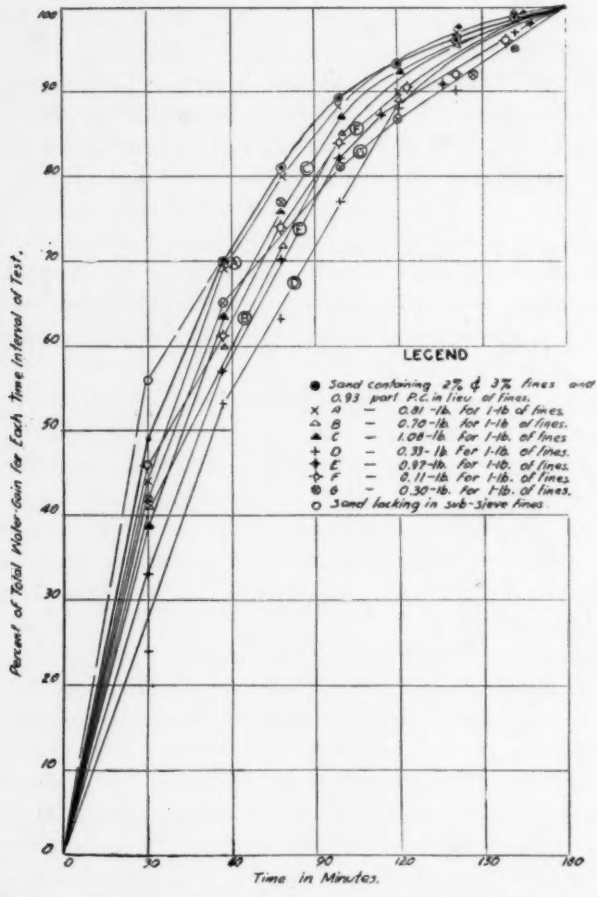


FIG. 3—RATE OF WATER-GAIN FOR EACH ADMIXTURE IN CONCRETE

few pounds of fines per cubic yard of concrete would prove expensive.

Fig. 4 shows some comparative costs of supplying fines for use in concrete in this District. It was found that there was about 1.3 per cent fines in the natural sand which was commercially available passing the No. 100 screen. In one instance, the cost of producing the other 0.7 per cent, required to bring the quota of fines to the 2 per cent specified, was 7½ cents per cu. yd. of concrete. This cost is not to be considered as typical. The cost of the materials required to produce bleeding action in concrete, equal to that of the specified 2 per cent fines are plotted in comparison with the cost of producing natural fines. The curves are self-explanatory and are subject to direct reading. For instance, if 0.3 per cent natural fines are lacking and it is

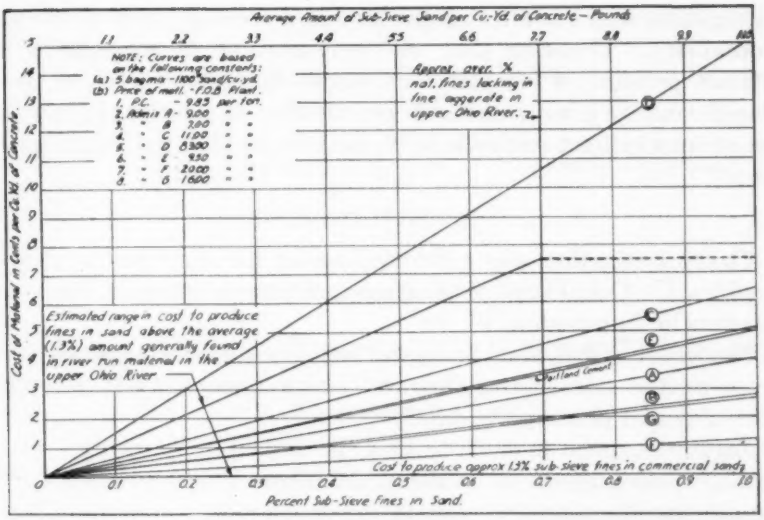

Average Amount of Sub-Sieve Sand per Cu. Yd. of Concrete – Pounds
NOTE: Curves are based on the following constants:
(a) 5 bags/cu. yd.
(b) Price of matl. - F.O.B. Plant.
1. P.C. - 9.83 per ton.
2. Items A - 9.00 " "
3. " B 7.00 " "
4. " C 11.00 " "
5. " D 8.300 " "
6. " E 9.50 " "
7. " F 2.900 " "
8. " G 1.600 " "
Report aver. % nat. fines lacking in fine aggregate in upper Ohio River.
Estimated range in cost to produce fines in sand above the average (1.3%) amount generally found in river run material in the upper Ohio River.
Portland Cement
Cost to produce approx 1.3% sub-sieve fines in commercial sands
Cost of Material in Cents per Cu. Yd. of Concrete
Percent Sub-Sieve Fines in Sand

FIG. 4—COMPARATIVE COST OF SEVERAL ADMIXTURES USED IN LIEU OF NATURAL FINES LACKING IN FINE AGGREGATE

decided to use admixture "A" for replacement, the cost of this material (see Fig. 4) will be 1¼ cents per cu. yd. of concrete. The quantity of admixture A required would be as follows:

$$3.3 \times 0.81 = 2.67 \text{ lbs.}$$

As implied in Fig. 4, the values shown apply only for the condition of mix design, the f. o. b. manufacturers' prices quoted and river condition indigenous to this District.

DISCUSSION

Since bleeding action in concrete seems to result, at least in part, from segregation, it is suggested that some such test as described in the foregoing might be made in conjunction with a mobility test for measuring comparative workability in concretes. In the final analysis it is believed that if a concrete is mobile and does not show water-gain, it may be considered as having good workability. Powers[4] gives this definition for workability as follows: "Workability is that property of a plastic concrete mixture which determines the ease with which it can be placed and the degree to which it resists segregation."

Pearson[5] says "More attention should be devoted to the study of

[4] "Studies of Workability of Concrete," by T. C. Powers, JOURNAL, Amer. Concrete Inst., February 1932, *Proceedings*, Vol. 28, p. 419 (1932).

[5] "Report on Significance of Tests of Concrete and Concrete Aggregates," by J. C. Pearson, A. S. T. M., 1935, p. 38.

segregation, which is after all the final and necessary test of adequate workability. Granting that the measurement of segregation is a difficult problem, a complete analysis of the constituents of freshly placed concrete is not necessarily required for its solution; in fact there are various indirect methods which may be simpler and less subject to error."

ACKNOWLEDGMENTS

This investigation was conducted under the direction of John F. Conklin, Lt. Col., Corps of Engineers, District Engineer, with headquarters in Huntington, W. Va.

The writer wishes to make special mention of the valuable assistance given by the staff of the District Testing Laboratory in compiling, analyzing and plotting the data.

Acknowledgment is made to Prof. R. E. Davis, University of California, whose suggestions furnished the necessary stimulus for starting ing this investigation.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for January-February 1937. Discussion should reach the Institute Secretary by December 1, 1936.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY, 1937

Discussion of a paper by John C. Sprague:

Evaluating Fines in Concrete on a Bleeding Test Basis*

E. W. SCRIPTURE, JR.†

This paper presents enlightening data on a comparatively little studied subject. Interest in bleeding and water gain appears to have developed recently and consideration of the relation of fines to these phenomena have not hitherto been adequately treated.

Mr. Sprague has made a study which reveals certain relations but he has by no means exhausted the subject and it was not, presumably, his intention in presenting this paper to present his conclusions as final. In studying the data given in the paper, therefore, certain other factors involved should be borne in mind.

One factor of considerable importance not considered, is the influence of fines on consistency. The paper states: "A cement factor of 1.25 and a water-cement ratio of 0.827 were used throughout, as these values represented an average field condition." It has been shown[1], however, that the addition of certain types of fines, such as some diatomaceous earths, will decrease the mobility as defined by slump or flow tests at a given water-cement ratio. On the other hand, it has been maintained though less conclusively proven that certain other fines, notably precipitator ash, even in properly designed mixes will lower the water-cement ratio required for a given consistency. Again, it has been shown[2] that certain other materials, not fines at all, will increase mobility at a given water cement ratio. Finally it is not clear from the paper whether, when portland cement was used as a fine addition, the water content was increased to maintain the water-cement ratio.

*JOURNAL, Amer. Concrete Inst., Sept.-Oct. 1936 p. 29.
†Director Master Builders Research Laboratories, Cleveland, Ohio.
[1]Powers, JOURNAL Amer. Concrete Inst., Feb. 1932, *Proceedings* Vol. 28, p. 419.
[2]Kennedy, *J. Ind. Eng. Chem.* Vol. 28 (1936).

It should be fairly evident that if the consistency of a mix is changed by any means whatsoever from, for example, a 4-in. slump to one of 2-in., the bleeding and water gain characteristics will be entirely different and the drier mix will show less water gain. On the other hand, if the slump is increased from 2 to 4 in. the water gain will be greater.

Putting together the postulates (1) that various admixtures affect the consistency of a given concrete mix differently and (2) that the bleeding characteristics of a given concrete mix will vary with the consistency: then it is obvious that the addition of materials which decrease slump will show greater reductions in water gain and those which tend to increase slump will show smaller reductions in water gain if tested at a fixed water-cement ratio than would be the case if they were tested at a fixed consistency. This immediately raises the question whether the effects observed by Mr. Sprague are due to differences in the influence of the various fine materials on bleeding or to their influence on consistency. It is obviously useless to reduce water gain by drying up the mix with an absorbent material if, in order to secure placeability, it is necessary to add sufficient water to bring the mix back, to its former consistency. Reducing this proposition to the absurd, substitution of slag for the relatively non-absorbent aggregate used would probably have produced a concrete which would have been so dry that it would have shown little or no water gain but also would have been entirely unplaceable. An absorbent fine material such as diatomaceous earth would have had a similar effect in a smaller degree. Incidentally it might be mentioned that with an absorbent aggregate or absorbent fines, the water-cement ratio calculated from the volumes of cement and water added to the mix is not the actual water-cement ratio since the absorbed water should be subtracted from the volume of water added.

Brown[3] has shown that bleeding is dependent, at least in part, on the characteristics, chemical and physical, of the cement and Kennedy[2] has shown that bleeding can be controlled by additions to the cement, which do not actually add to the solids of the mix but act by altering the state of the cement. In view of these findings it becomes a question whether the control of bleeding is not to be sought in the cement itself rather than in the addition of fines.

Mr. Sprague sums up by saying that "if a concrete is mobile and does not show water gain, it may be considered as having good workability." While this may well be correct he does not relate water gain as affected by additions of fines to mobility. At the same time

[3]Levi S. Brown, *J. Ind. Eng. Chem.* Vol. 27 (1935) No. 1 p. 97.

he suggests a relation between water gain and segregation. It would seem more probable that segregation is more directly related to excessive mobility and that water gain may or may not accompany high mobility. That bleeding may occur without appreciable segregation, implying by segregation an unequal distribution of coarse aggregate, fine aggregate and cement in the hardened concrete, is shown by Brown. He shows also that water gain as an external manifestation of bleeding may be an index to bleeding characteristics but does not present a complete picture of the phenomenon. Perhaps a result of bleeding as serious as or more serious than the water gain is the accumulation of water under aggregate particles which impairs the bond between cement and aggregate and develops planes of weakness in the concrete. It is at least conceivable that this result of bleeding may occur without either segregation or visible water gain.

With respect to the computations of costs for different materials required to produce a given bleeding action, mobility is again not taken into consideration. Furthermore none of the other properties of the concrete as affected by the various fine materials are evaluated. As is stated by McMillan[4] and Powers, "It follows, therefore, that any beneficial effect which an admixture may have must compete with the detrimental effect of any increase in water content which its use may incur. This holds for portland cement as well as for other powdered materials." Later in the same paper they state: "The real worth of an admixture should be determined on the basis of its effectiveness in producing, under definite curing conditions, the specific properties desired in concrete. This may require data on its effect on strength, workability, volume change, resistance to disintegration by freezing and thawing, and resistance to the action of strong sulphate waters."

These comments are offered, not to criticize a valuable contribution to a neglected subject, but to point out that the limited data presented in the paper should not be interpreted and applied indiscriminately without considering the many other variables involved and not falling within the scope of the paper. In fact, in a discussion of a paper on workability,* Mr. Sprague points out some of these very considerations. In a series of tests on concretes using different aggregates he determines bleeding at equal mobility but mentions that in tests to evaluate the effect of sub-sieve fines on bleeding action, presumably referring to the data of the present paper, no attempt was made to control the mobility of the concrete. He further suggests that a test

[4]McMillan & Powers, JOURNAL Amer. Concrete Inst., Mar.-Apr. 1934, *Proceedings* Vo. 30, p. 325.
*Herschel & Pisapia, JOURNAL Amer. Concrete Inst., May-June, 1936, *Proceedings* Vo. 32, p. 641.

which combines a measure of mobility and of bleeding action might serve as a measure of workability. It is to he boped that the further series of tests in this connection which he mentions will throw further light on the subject. The cost of fine material should not be determined on a basis of its effect on water-gain at a fixed water-cement ratio alone, but as well on its effects on various other properties pointed out above. Finally, the question arises whether bleeding and water-gain can or cannot be more effectively and economically controlled by means other than the addition of fines.

AUTHOR'S CLOSURE

Dr. Scripture's point that the influence of fines on consistency of concrete should be considered in making a study of these same fines on bleeding action of mixing water is well taken, and was considered by the writer at the time these tests were made, and in later tests. It is unfortunate that no mention of this factor of workability was made in the paper.

In tests made to compare the consistency of concrete containing the various fines included in the paper the water-cement ratio was increased to 0.94 for all mixtures. This was done to insure unrestricted flow in the modified remolding test used to measure the mobility of concrete. Concrete slump tests and mortar flow tests were also made. The remolding test used involved dropping the top of the water-gain table a sufficient number of times to cause the plastic concrete to assume a cylindrical shape conforming with that of the container; the number of drops required to bring about this change in the shape of the concrete, from that of a truncated cone, was considered as the measure of remolding effort.

By holding the water-cement ratio constant for all mixtures, and varying the quantities of different types of fines, it was found that the bleeding action was controlled by the same factors brought out in the original paper. On the other hand, it was found that the admixtures produced concrete having less mobility than the control mixture.

Table 3 shows the results obtained in making this comparison. Each value is the weighted average of several tests.

TABLE 3—COMPARISON BETWEEN MOBILITY AND BLEEDING ACTION OF CONCRETE

Material Designation	Remolding (No. Drops)	Bleeding (%)	Slump (in.)	Mortar Flow (%)
2% Natural fines	19	12.7	4½	19.5
No fines	29	13.7	3	21.5
A	25	12.7	3½	22.0
B	28	12.7	3	20.0
F	33	12.4	2½	22.5
G	28	12.5	3	22.0
Portland cement	25	12.5	3½	20.5

It will be noticed that all the mixtures are less mobile than the control mixture (2 per cent nat. fines). On the other hand, the flow of the mortar entering these mixtures was found to vary only slightly from that of the control. Except for the mix containing no fines, the trend of the flows followed in the same direction as that of the bleeding tests.

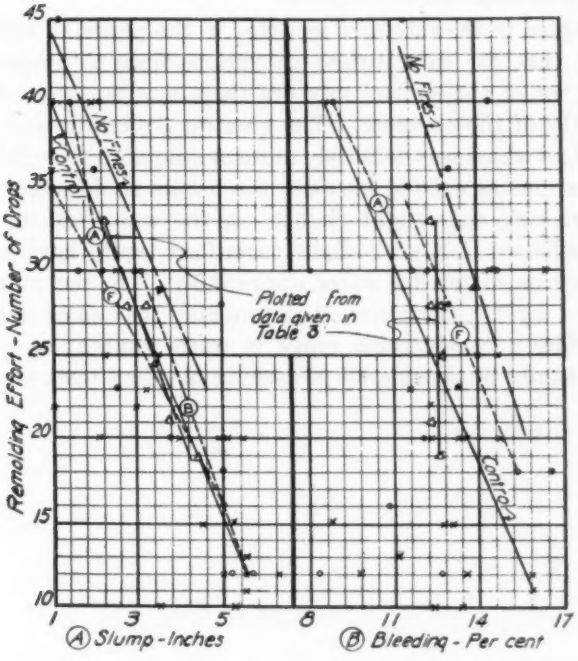


FIG. 5—RELATION OF SLUMP TO REMOLDING EFFORT; ALSO BETWEEN PER CENT BLEEDING AND NUMBER DROPS REQUIRED TO REMOLD CONCRETE

Note that for curves plotted from data given in Table 3, the range in slump is from 2¼ to 4¼ in. and the remolding effort from 19 to 33 drops, for different mixtures having approximately the same per cent bleeding. Each point represents several tests.

The curves in Fig. 5 bring out the relationship between mobility of concrete and bleeding action more clearly than does the table. Tests on which Fig. 5 is based were made using 2 per cent fines in each mixture and holding the water-cement ratio constant. This caused bleeding and mobility of the mixtures to vary over a large range. Two of the curves are for mixtures having approximately equal bleeding action and are based on values obtained from Table 3. In the left hand side of the figure, slump is plotted against remolding

effort and it will be noticed, particularly for the control curve, that there is a fairly definite relationship between these two measures of mobility. As the slump increases, the effort required to change the shape of the concrete decreased. In the right hand side of the figure the relationship between bleeding action and remolding effort is shown. Individual tests show erratic results, although it was possible to draw an average curve for them.

With regard to the mixtures reported in the paper, it will be noticed (Table 3) that for substitution of various fines to produce bleeding equal to that of the control mixture there was no great difference in the mobility of the concrete. The greatest deviation was in the case of the control which was definitely more mobile than the other mixtures. Such variations as existed between the others could have been coped with by making only slight changes in the water-cement ratio, and it is not believed that such slight variations would have affected the bleeding action of mixing water appreciably. On the other hand, as pointed out by Dr. Scripture, it is probable that some fine materials might require a considerable range in water content and this factor should certainly be considered in making bleeding tests of concrete.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . SEPTEMBER-OCTOBER 1936

Report of Committee 107

Properties of Job-Cured Concrete at Early Ages

HARLAN H. EDWARDS, AUTHOR-CHAIRMAN

In the preparation of this report, Mr. Edwards had the cooperation of the critic members of his committee: R. W. Crum, H. E. Davis, T. E. Stanton, M. A. Timlin, M. B. Lagaard, C. A. Hughes and Frank L. Howard. Completed in December 1935, its publication has been unavoidably delayed. If the Committee does not succeed in establishing a correlation of the results of cylinder and beam tests with actual job quality, it does make out an impressive case for the multiplicity of factors which, under customary job practice, set up innumerable stumbling blocks to the desired correlation by which the actual quality of the concrete in the structure is to be known.—Secretary A. C. I.

ABOUT ten years ago during the California winter weather, a batch of test cylinders from a building job were cured in the field as provided by the standard of the A. S. T. M. The results were so low that it was determined to find out why, and whether the concrete in the structure was really of as low strength. The next time cylinders were made, duplicates were provided, one set being cured as before, while the other set was taken to a laboratory the next day and cured in the standard 70° damp condition. The results were so strikingly different that other tests were made to compare concrete in the structure with test cylinders, as reported to the Institute in 1929.*

Standard test cylinders frequently did not reflect the physical characteristics of the job. Tested cement, inspected aggregate, specified proportions and adequate mixing were not all. Undercuring, with its prematurely dried-out concrete, coupled with cloudy, cool days and nights below freezing, produced a structure having character-

*"Progress in Determining the Relation between Test Cylinders and Concrete in the Structure," by Harlan H. Edwards, JOURNAL, Amer. Concrete Inst., November 1929; *Proceedings* Vol. 26, p. 57.

istics far from those indicated by laboratory-cured test cylinders. Fortunately, the water-cement ratio law was operating long before its true significance was discovered. Concrete placed as "soup," unless hopelessly segregated, was improved somewhat by the action of wood forms in absorbing some of the excess water, by permitting more water to escape through cracks or joints between boards, or to be drawn off the slabs during finishing operations. Thus the concrete in the structure, by reducing the water-cement ratio before time of set, became a little better.

Just how much these various factors affected the mass in commercial work was not known. Test cylinder reports were filed away; occasionally a piece of the structure was removed for testing, but no correlation between the original concrete as placed, and this tested piece was possible. Time and temperature, moisture and placing conditions all cooperated in producing that characteristic which the test disclosed, yet none of them could be separately evaluated unless they were definitely controlled and measured.

SCOPE OF ACTIVITY OF THE COMMITTEE

The scope of the study was enlarged with the formation of Committee 107—instructed to summarize available information as a basis for safe formwork removal or road opening—information which would relate the properties of job-cured concrete at early ages to control cylinders, laboratory-cured; to discuss the effects of curing conditions, water-cement ratio and aggregates, involving considerations in the construction of roads and buildings at different temperatures.

On all assignments, so broad, limits were set at the start as economic conditions reduced the active personnel and limited the time available. Individual research has been so curtailed that data contained herein are fewer than originally anticipated. It is hoped that through the voluntary cooperation of others interested but not now known to the committee, the work can be carried forward more rapidly and comprehensively.

Data are needed by which, having a continuous record of the weather during the curing period, and of the job characteristics, the results of the standard 70° cured cylinders or beams at any age may be quickly translated into terms of the approximate condition of the concrete on the job.

EXISTING DATA MEAGER

Search has been made in available literature for pertinent data. With few exceptions the result is expressed by M. A. Timlin:* "The work which has been done in this direction seems to be woefully meager, and it appears that the committee's reports will need to be based on original research."

As to information on highways, T. E. Stanton† reports a similar lack of data: "Most of the work done has been on investigations of concrete curing by different methods. In these investigations very little effort was made to compare job and laboratory curing at early ages." Many data are at hand of test cylinders field-cured, or of similar core and beam tests, but to correlate these with each other or with standard tests, no curing temperature or subgrade record is available with the aid of which the one can be translated in terms of the other. Successive field-cured cylinders generally have no known relation one to another because of the different curing and testing conditions (whether tested moist, dry, or partly dry, or at temperatures other than standard), hence they form a record worth little except as a general guide to quality.

MATERIALS AND TESTING

Except as otherwise noted, the limited tests mentioned in this report were made under the direction of the author-chairman as a part of his personal research program. In general, the same methods and materials were used as previously described,‡ though on tests made on the San Diego, Calif., Museum of Natural History, Otay rock and sand of similar size-characteristics were used. Riverside cement was used on all series but that of the Progress-Bulletin building in Pomona, Calif., on which Colton cement was used. All cement, and concrete test specimens were tested by the Raymond G. Osborne Laboratories or the Smith-Emery Co., Los Angeles, in cooperation with the Committee. Water-cement determinations were from analyses of wet concrete by the Dunagan apparatus.

WALL TESTS

As indicated in the Claremont tests‡ the general trend of strengths in well-cured walls is higher than shown by the control cylinders, though there is a little variation in the strengths of the sawed prisms of the different sets, probably due to the variations in effective curing and to temperature during the curing period. Control cylinders cast

*Testing Engineer, E. L. Conwell Co., Philadelphia, Pa.
†Materials and Research Engineer, Division of Highways, State of California, Sacramento, Calif.
‡JOURNAL, Amer. Concrete Inst., November 1929, p. 57-64.

TABLE 1—SUMMARY OF TEST DATA

Test No.	Date Made	Building	Size of Test Piece	Method of Curing	Place of Curing	Slump 1 inch	W/C	Mix	F. M.	Lab. Cured Control Cylinders 7d.	Lab. Cured Control Cylinders 28d.	Cyl. Concrete "As Placed" Cast and Cured in Wall 7d.	Cyl. Concrete "As Placed" Cast and Cured in Wall 28d.	Taken from Form 1 to 2 hr. Later Lab. Cured 7d.	Taken from Form 1 to 2 hr. Later Lab. Cured 28d.	Same Cast and Cured in Wall 7d.	Same Cast and Cured in Wall 28d.	Block of Concrete Cured in Structure 7d.	Block of Concrete Cured in Structure 28d.	Moist Condition at Test
403 to 414	5/24/29	Scripps College Unit 3	6 x 12	In 8-inch Wall	N.-S. Wall	8 and 2	—	1:4:4 Damp Loose	—	—	1604	—	2225	—	2257	—	2263	—	2565	Partly Dry
425 to 430	12/18/29	Unit 4	6 x 12	In 8-inch Wall	S. Wall	6 to 7	—	1:3:3 D. L.	—	—	2457	—	3386	—	—	—	—	—	2950	Dry
458 to 471	1/22/30	Unit 4	6 x 12	In 8-inch Wall	N. Wall	6	—	1:3:3 D. L.	—	—	3365	—	2855	—	3834	—	3150	—	3063	Wet
464B to 466B	1/22/30	Unit 4	6 x 12 4 x 4 x 4	In 4-inch Slab	2nd Floor	6	—	1:3:3 D. L. Same Batch	— —	— —	3365 —	— —	— —	— —	— —	— —	— —	— —	— 3266	Wet
487 to 510	2/18/30	Unit 4	6 x 12	In 8-inch Wall	N. Wall S. Wall	3 and 1	—	1:3:3 D. L.	5.61	1956	2523	—	3397 3296	—	2724	—	3104	—	2531 2834	Wet
487 to 510	2/18/30	Unit 4	6 x 12	In 8-inch Wall	E. Wall W. Wall	3	—	1:3:3 D. L. Same Batch	5.61	1956	2523	—	3092 2708	—	—	—	—	—	2775 2412	Wet
472 to 477	1/23/30	Unit 4	6 x 12 4 x 4 x 4	In 4-inch Slab	2nd Floor	6-7	—	1:3:3 D. L.	— —	1581 —	2332 —	— —	— —	— —	— —	— —	— —	— 2114	— 3109	Wet
511 to 534	4/ 2/30	Pomona College Men's Dorm.	6 x 12	In 12-inch Wall	N.-S. Wall S. Wall	5	—	Inundated 1:28:3.17	5.36	1936	3340	—	3585 3542	—	—	—	—	—	3817 3821	

Test No.	Date Made	Building	Size of Test Piece	Method of Curing	Place of Curing	Slump 1 inch	W/C	Mix	F. M.	Lab. Cured Control Cylinders 7d.	Lab. Cured Control Cylinders 28d.	Cyl. Concrete "As Placed" Cast and Cured in Wall 7d.	Cyl. Concrete "As Placed" Cast and Cured in Wall 28d.	Taken from Form 1 to 2 hr. Later Lab. Cured 7d.	Taken from Form 1 to 2 hr. Later Lab. Cured 28d.	Same Cast and Cured in Wall 7d.	Same Cast and Cured in Wall 28d.	Block of Concrete Cured in Structure 7d.	Block of Concrete Cured in Structure 28d.	Moist Condition at Test
511 to 534	4/ 2/30	Pomona College Men's Dorm.	6 x 12	In 12-inch Wall	E. Wall W. Wall	5	—	Inundated 1:2.8:3.17 Same Batch	5.36	2 x Mo Cyli 2318	4 rtar nder 3987	—	3410	—	—	—	—	—	3615	
													3405						3527	
661 to 701	1/16/31	Claremont Colleges Auditorium	6 x 6 6 x 6 6 x 12	In Slab Sand — Cover Sisalkraft Cover	1st Floor	5 to 6	.995	4 Sacks 1161-lb. S. 1064 lb. g.	5.38	1761	2911	—	—	—	—	—	—	1256	2620	Wet
																		1586	2492	
805 to 826	7/22/31	Harper Hall	4 x 4 6 x 12	In Slab Sand Cover Sisalkraft Cover	1st Floor Sun	4	.966	1:2.87:3.13 D. R.	5.45	1940	2623	—	—	—	—	—	—	2105	3057	Wet
																		2480	2660	
839 to 862	7/22/31	Harper Hall	6 x 12	In 8-inch Wall	N. Basement Wall	5 to 6	—	1:2.87:3.13 D. R.	5.43	1562	2377	1342	1945	—	—	—	—	1505 H Wea	2085 ot ther	Wet
930 to 955	12/26/31	Progress-Bulletin Bldg.	6 x 12 6 x 6	In 8-inch Slab	3rd Floor	7	—	1:3:3 D. L.	—	1340	2480	—	—	1780	3090	—	—	2225	3461	Wet
			Same	Batch, Cy linders Cur ed			on S lab.						1180	—	—	1508	2925	—	—	Wet
M. A. 1/24/	Timlin 31	——	6 x 12	In 7-inch Wall	——	6	1.08	1:2.12:4.7 D. R.	—	—	3587	Forms Off	2730					Av. Max. Temp. 42° Av. Min. Temp. 27°		
			Cylinder s Cured Be side Wall o n Jo b								989	Forms On Forms Off	2841 2663	Wet Once Daily		...		Temp. conc. when placed 45° Tested cold—below freezing		

and cured in walls show also a somewhat erratic trend, though they are generally slightly lower than the prisms cured adjacent. It is believed, however, that walls in buildings are usually of somewhat lower strengths due to the few days of curing usually given them. Where forms are left on and occasionally sprinkled, however, moisture remains and produces fairly satisfactory curing, as illustrated by the results from the Pomona College Men's Dormitory test in Claremont, which was run in hot summer weather. (Table 2.)

Here two different 12-in. walls, south exposures, 20 ft. apart, were used (to check accuracy) together with an east and a west exposure nearby, to ascertain directional effects. No north exposure was available. Three cylinders were cast in a block of concrete in each exposure, together with a block of concrete from which three prisms were later sawed prior to test. Note the close correlation between similar specimens.

TABLE 2—WALL CONCRETE STRENGTHS
POMONA COLLEGE MENS DORMITORY
Strength (p. s. i.) of 29-day Specimens Cast in Walls

	South No. 1	South No. 2	East	West
Job Cured Cylinders	3650	3560	3438	3414
	3646	3616	3471	3276
	3459	3449	3321	3516
Average	3585	3542	3410	3405

Quality Cylinder	7 days 1939	29 days 3340	Average Strengths
Quality Mortar Cylinder	7 days 2341	29 days 3922	Average Strengths

Sawed-out Prisms	3731	3981	3756	3420
	3891	3887	3733	3110
	3830	3594	3357	4052
Average	3817	3821	3615	3527

Curing Temperatures Degrees Fahrenheit

	Air	Concrete
1st week	48 to 90	66 to 74
2nd week	50 to 80	62 to 74
3rd week	48 to 95	65 to 84
4th week	52 to 82	61 to 77

Data on Mix Proportions 1:2.83 : 3.17 by vol., sand inundated
1:3.07 : 3.3 by weight
2½-lb. Celite per sack. Slump 5 in.
Fineness Modulus {Sand 3.45, Rock 7.17, Concrete 5.36}
Average weight control cylinders 145.9, prisms 147.16 lb. per cu. ft.

Probably the most interesting of the various series of tests on wall concrete, however, was made during the construction of the Museum of Natural History in San Diego, in June, 1932. Two sections of wall 8 in. thick, 52 in. long, 48 in. high were cast in wood forms with non-absorbent prestwood-lined bottom and end bulkheads. Weak-planes

were created by inserting a mat of reinforcing steel rods where separation was desired, breaking the bond by moving them as the concrete hardened, so that vertical sections of the wall could be removed easily. Quality cylinders were cast of the same concrete and cured under sisalkraft adjacent. After two days in moderately warm weather the sections were removed from the walls and carefully transported, together with the control cylinders, to the curing room of the Osborne Laboratory in Los Angeles, the entire operation being completed in about 6 hours. Subsequent curing was carried on there under standard conditions. Test pieces were cut by a wire-carborundum cutting machine in the laboratory, and the specimens returned immediately to the curing room. Due to the extreme hardness of the rock, samples could not be cut in time for a 7-day test, so 28-day tests only were obtained, which were: Quality Cylinders 2021 p. s. i. Sawed prisms 2994 p. s. i. (Test sections of floor slab were also made and are reported under "Slab Concrete.")

Test cylinders cast in walls and also stored alongside walls in freezing weather by M. A. Timlin on work in New York City showed rather uncertain results, yet analysis discloses the reason. The data follow:

Proportions 1:2.12:4.70 dry rodded measure. F. M. sand 2.63; rock 6.58; 6-in. slump. w/c (vol.) 1.08.

Temperature of concrete when placed, 45°F.; ½ hr. later 40°F.; maximum curing temperature, 58°F., minimum, 17°F. Average 42 to 27. Quality cylinders, laboratory-cured, 28 days, 3587 p. s. i. Air-cured cylinders beside wall, 28 days, 989 p. s. i. Ice throughout. Cylinders cured in 7-in. wall, forms left on, 28 days, 2841 p. s. i. Cylinders cured in 7-in. wall, forms removed, 28 days, 2730 p. s. i. All job cylinders were tested cold as they came from job; partially dry instead of being immersed 48 hr. in 70° water. When broken, the cylinders stored beside the wall contained frost crystals throughout, while crystals were found only in the outer inch of the wall. Testing dry or partly so gave an erroneous result also, while testing either cold or in a frozen condition accounted for the rest.

EFFECT OF TEMPERATURE AT TIME OF TEST, UPON STRENGTH

Since the preceding results seem at variance with normal experience, let us see the reason therefor. The relation between strength of portland cement mortars and temperature at the time of test was demonstrated by work done at the University of Texas* in which tension briquettes, 2 x 4-in. cylinders and 3 x 4 x 26-in. beams of various mixes were stored under standard conditions, then cooled or heated to the

*University of Texas Bulletin 2825, July 1, 1928. Eng. Res. Series No. 26.

desired temperature immediately before testing. Results showed that there was a marked change in strength with the change in testing temperature; an increased variation in strength occurring as the cement content increased. Although for test temperatures above 70°F. the strength decreased materially, lower temperatures increased the strength to a smaller degree. This change in strength is also shown by student thesis work at the University of California and at the University of Illinois, in which the trend of variations in strength approximates slightly less than 0.2 per cent decrease in strength for each degree of temperature rise above freezing. Below freezing, the strength of wet concrete rises rapidly, the amount perhaps being dependent upon the amount of water present and available for the formation of ice. Modulus of rupture tests in the Texas work showed a drop of approximately 80 per cent in the range between freezing and boiling, or approximately 0.45 per cent per degree F.

Contradictory evidence is given by Hogentogler* in analysing data on field-stored cylinders and beams for highway concrete. It is not believed, however, that this is conclusive in character, for the differences in modulus of rupture between 28 and 180 days are not large, and while he is discussing temperature *at the time of test*, it seems obvious also that the temperature in his tests affected the curing over a considerable period, since the cylinders and beams referred to in his paper were apparently *field stored*.

It is evident, therefore, that in all strength tests of portland cement, the temperature at the time of test should be closely controlled, since the results are of value only when compared with standards. From preliminary work in Texas, it seems that there is an approximate time-recovery of original strength upon return to original temperature, though the effect of repeated variation of temperature is at yet uncertain. It is hoped that added research may be carried on in this field.

FLOOR SLAB CONCRETE

Some of the wall test series included specimens of monolithically-finished floor slabs placed of the same concrete as the wall specimens by job workmen, who gave the test slabs the finishing treatment received by the floors on the job. The test on the Progress-Bulletin Building in Pomona, Calif., was typical of slabs having the dusted-on monolithic finish, and included the preliminary tamping of the slab to force down the coarse aggregate, the later drawing-off of the excess water and laitance, the application of 1:1½ dry mix floated in, and the various trowelings as done on the slab adjacent. These slabs were cast

Proceedings, 11th Annual Meeting of Highway Research Board, 1931, p. 279-80.

in openings which were to be used for ventilation ducts and were made at a time when days were warm and the night temperatures dropped nearly to freezing. The forms had wood bottoms and non-absorbent sides. The specimen thickness was the same as the floor slab in all cases. (Table 3.) Slab 8 in. thick, mix 1:3:3 loose measure; 7-in. slump, 6-in. cubes cut for test.

TABLE 3—SLAB CONCRETE, PROGRESS—BULLETIN BUILDING

Strengths in p. s. i.

Age	Standard Cylinders	Cylinders Cured on Slab	Slab Concrete Cast in Cylinders 1½ Hrs. After Placing in Floor		Slab Strength Sawed Cubes
			Cured in Lab.	Cured on Job	
7 d.	1340	1180	1780	1508	2225
28 d.	2480	——	3090	2025	3461

The tests previously mentioned under "Wall Concrete," made in San Diego, included also sections of slab cast and finished as described, on a prestwood panel form. These, made of the same batch of concrete and cured under the same temperature and moisture conditions as the wall sections and quality cylinders showed 28-day strengths corrected to cylinder H/D equivalent, of 3140 p. s. i. For comparison, wall test was 2994 p. s. i., quality cylinders, 2021 p. s. i.

Work done in Minneapolis, Minn., on cement floor topping laid by the Kalman process under the direction of Prof. C. A. Hughes of the University of Minnesota, adds further testimony to the fact that in mortar, as in slab concrete, the strengths in the structure seem to be greater than the standard tests of the material "as mixed"—provided, always, that curing has been thorough and at *normal* temperatures.

Effect of *high* curing temperatures was shown by a few slab tests run in Claremont, Calif., during weather which frequently approached 100°F. Since the test block in the floor was exposed to the full heat of the sun, the concrete temperature reached 100° or more. Contrary to previous tests, strengths of the sawed-out specimens, instead of being definitely higher, were only equal to or less than those of the quality cylinders. This was in accord with Boulder Dam tests, and with tests of the Missouri State Highway Department on road slabs, both of which showed that the strength of a concrete slab cured at a temperature of approximately 100°F. is lower than that of similar concrete cured at 70°F.

It is seen, therefore, that the strength of monolithically-finished slab concrete is invariably higher than that of the comparable concrete in walls. If curing has been properly done, with due allowance for temperature, the same should be true also of the relation to control cyl-

inders. Removal of water by (1) percolation through forms, (2) absorption into them, (3) evaporation, (4) by compaction of the mass, (5) by subsequent drawing off of excess water, if any, rising to the top, and (6) by its absorption into the dry-mix applied before troweling, reduces the water-cement ratio to create a stronger, denser concrete.

SLAB CURING UNDER DIFFERENT MEDIA

Efforts to determine the relative merits of damp sand and kraft paper for slab curing brought about a duplicate series of tests, whereby slabs were cured side by side on the job, one covered with 1½-in. of damp sand, the other covered with sisalkraft, cemented at the edges. The temperature of the sand-covered slabs were consistently lower than the paper-covered specimen by from 1° to 6°F. during the curing period.

TABLE 4—COMPARISON OF AIR TEMPERATURE WITH SLAB TEMPERATURE

Air	Sand Cover	Slab Below Sand	Slab Below Sisalkraft
63	57	53	59
55	50	49	53
67	58	58	61
56	45	48	50
64	56	56	56
70	55	53	55

The records of Table 4 are from thermometer readings at irregular intervals. In both series (Table 1—Tests 661 to 701, 805-826) the 7-day test on sawed-out cubes showed Sisalkraft-cured concrete highest, while in the 28-day test this was reversed. It was evident that the paper curing did not cool by evaporation as did the sand, so the early result was higher. On the other hand, since the paper did not cover the lower side of the slab and beams, nor supply additional curing water from above, the dry, warm summer air of the arid climate removed the moisture from below, drying out the slab and causing the normal increase in strength to cease. This demonstrates the fallacy of comparing job-curing media by encasing entire test specimens, such as cylinders, in paper bags, or coating entirely with asphalt, etc., for this condition is seldom, if ever, found in practice. Adequate curing requires the continued presence of moisture supplied by repeated wettings of all parts of the structure, the necessary frequency of which is dependent upon weather conditions and upon the moisture-supplying or retaining nature of the covering material, while the duration depends upon the air temperatures and upon the character or constituents of the cement used.

It would appear that good judgment must be used in the selection

of a curing medium to suit the weather. As found on the Missouri State Highway tests, slab temperatures under paper, and more particularly under dark-colored curing agents become extremely high in hot weather, causing excessive cracking as well as lower strengths. In such times, either water, wet sand, or white-colored curing agents should be used. Conversely, the darker-colored curing agents may be of material advantage in cooler weather in accelerating hardening by keeping the slab temperature higher.

STRENGTH VARIATION IN HEIGHT OF COLUMNS OR WALL

Tests of prisms cut from different sections of 10 x 10-in. plain concrete columns showed a definite trend toward reduction in strength from bottom to top. Planes of separation were created in the columns by the insertion of a mat of reinforcing rods across the form after placing the concrete, approximately 40 in. apart, which were moved during set so as to break the bond. Forms were removed after 4 days, and the columns cured with the rest of the structure. One column was removed for the 7-day and the other for the 28-day test. Unfortunately the columns were not situated where they could be shaded entirely from the sun, so the upper sections show a higher strength than normal due to this cause. In the structure, columns are entirely shaded, usually by the slab above.

TABLE 5—TEST COLUMNS, CLAREMONT COLLEGES BUILDINGS

Bridges Auditorium				Harper Hall	
Distance Above Base to Center of Specimen	7-day p. s. i.	29-day p. s. i.	Weight p. c. f.	7-day p. s. i.	Distance Above Base
7″	1676	3219	150.02	1600	6″
18″	1285	3282	147.99	1430	18″
30½″	1396	——	149.33	1525	30″
53″	1546	3684	149.75	——	——
66″	1408	2859	147.19	——	——
79″	1231	2853	148.52	2180	90″
120″	1027	2716	146.59	1890	102″
137″	1480	2602	148.82	1705	114″
Standard Quality Cylinders	1761	3098	——	——	——

JOB-CURED CYLINDERS VS. CONCRETE IN STRUCTURE

Many architects and engineers unthinkingly insist that a fair measure of job concrete strength is obtained from the results of tests of concrete cylinders cured alongside a wall or on the slab. Perhaps this is approximated in warm weather, though probably part of the reported strength is due to the increase caused by a dry or partially dry specimen which has not been immersed 48 hours prior to test, or due to a higher curing temperature than that existing within the larger mass of the structure. At other times, when air temperatures get down

below 50°F., or when specimens are cured in damp sand in the shade, such comparisons are apt to be entirely incorrect and misleading. Having a greater surface area exposed to the weather with much smaller volume, the curing temperature of the cylinder follows very closely that of the air. The temperature of the structural concrete, contrarily, follows a median course, neither as warm nor as cold as that of the air, though tending to approach the cold limit during repeated cold periods. Temperatures are kept up slightly during the early curing period by the heat of hydration of the cement. Structural vs. air temperature variations have already been shown in considering the tests on the Pomona College Men's Dormitory.

Likewise, moisture conditions in the cylinder and in the structure are unlike. The cylinder dries quickly after placing as well as after being wetted, while the structure, with its smaller comparative surface area retains water a much longer time. Other conditions previously described, such as the effect of forms, manipulation, evaporation, etc., also act to make such a comparison impossible. Mr. Timlin's job-cured cylinders (see under "Wall Tests") showed 989 p. s. i. as compared to 2730 p. s. i. from cylinders cast in the wall. Had the cylinders been previously immersed in water as required by standards, the difference would undoubtedly have been much greater. The difference in temperature in his work was also shown by the presence of ice crystals throughout his cylinders cured on the slab, while these crystals were present only in the outer inch of the wall. Similarly, the author's Progress Bulletin Building tests show 1180 p. s. i. in 7 days for the cylinders cured on the job in capped metal containers, while cubes cut from 8-in. slab specimen at the same age, corrected for the H/D relation, showed a strength of 2225 p. s. i., both exposed under sisalkraft to the same weather conditions side by side. During this time night temperatures were near 40°F., with a daytime maximum of about 60°F., creating a concrete temperature of 57° to 45°F.

CURING TEMPERATURE KNOWLEDGE IMPORTANT

It is commonly known that, except for certain variations in the region above 70°F., concrete hardens more rapidly as the temperature increases, and that concrete cured at low temperatures, above the freezing point, gains strength slowly. Work done at the University of Illinois in 1915 and by Duff Abrams in 1920 has been carried further by the Portland Cement Association* while recent research on cements in the Boulder Dam investigation at the University of California shows that the rate of increase in strength reaches a maximum in the neigh-

*"Temperature Effects on Compressive Strength of Concrete." A. G. Timms and N. H. Withey; JOURNAL, Amer. Concrete Inst., Jan.-Feb. 1934; *Proceedings*, Vol. 30, p. 159.

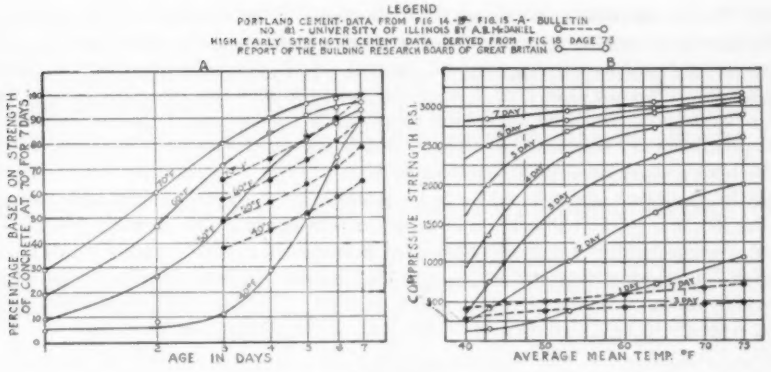

LEGEND
PORTLAND CEMENT- DATA FROM FIG 14 -B- FIG 15 -A- BULLETIN NO. 81 - UNIVERSITY OF ILLINOIS BY A.B. McDANIEL
HIGH EARLY STRENGTH CEMENT DATA DERIVED FROM FIG. 18 PAGE 73 REPORT OF THE BUILDING RESEARCH BOARD OF GREAT BRITAIN
A
PERCENTAGE BASED ON STRENGTH OF CONCRETE AT 70° FOR 7 DAYS
AGE IN DAYS
B
COMPRESSIVE STRENGTH P.S.I.
AVERAGE MEAN TEMP. °F

FIG. 1

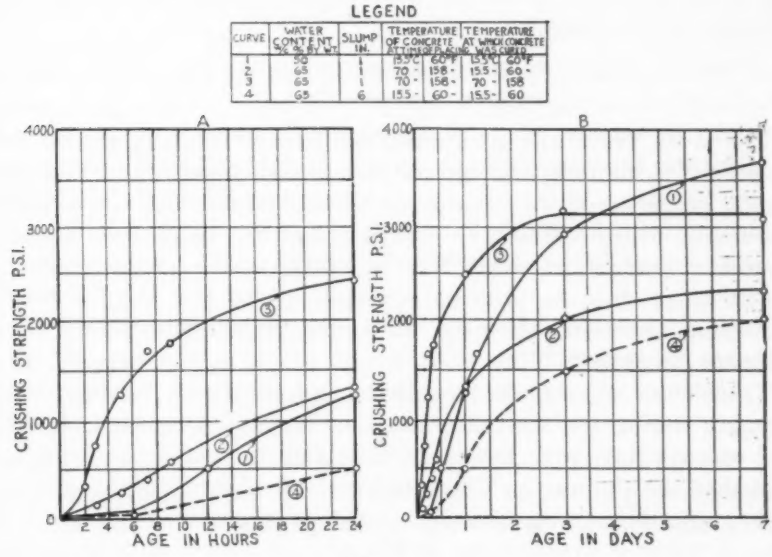

LEGEND
CURVE | WATER CONTENT % BY WT. | SLUMP IN. | TEMPERATURE OF CONCRETE AT TIME OF MIXING | TEMPERATURE AT WHICH CONCRETE WAS CURED
1 | 50 | 1 | 155° C 60° F | 155° C 60° F
2 | 65 | 1 | 70 - 158 - | 155 - 60 -
3 | 65 | 1 | 70 - 158 - | 70 - 158
4 | 65 | 6 | 155 - 60 - | 155 - 60
A
CRUSHING STRENGTH P.S.I.
AGE IN HOURS
B
CRUSHING STRENGTH P.S.I.
AGE IN DAYS

FIG. 2

borhood of 70° curing and that 110° curing produces a lower rate of increase and lower strength. Fig. 1, prepared before the P. C. A. work was done, shows the entire inadequacy of our present-time-temperature-strength data for use with modern cements, particularly those of the rapid hardening type. Results of British research in this field well sums up the situation:

(1) The investigation shows the importance of the maturing temperature of concrete test pieces made on the job. Without a knowledge of temperature conditions it is impossible to interpet the test results.

(2) The investigation also shows the importance of protecting small or thin concrete members such as columns, floors, and wall slabs from the action of the cold, when using rapid hardening portland cement. It is not only important to guard against frosts in such cases but, it would appear from the tests reported, that to take full advantage of the rapid hardening properties of the cement, it is also imperative that precautionary measures be taken to protect the concrete after it has been placed, if the temperature becomes lower than 50°F.

(3) In specifying strength values to be attained by concrete at early ages sampled on the job, it is evident that the maturing temperature must be carefully considered. Even the temperature range of 14.4°C. (58°F.) to 17.8C. (64°F.) as laid down in the British Engineering Standards Assn. specification for portland cement permits considerable variation in the early strength of concrete. Within this temperature range there appears to be a variation of 30 per cent in the strength at 24 hours.

Examination of the time-temperature-strength relations shown in Fig. 1 and 2, therefore, shows the inadequacy of previous work when applied to modern cements, and demonstrates further the need for new research on this point.

CEMENT CHARACTERISTICS AFFECTING STRENGTH AND HEAT PRODUCTION OF CONCRETE

The work previously mentioned on Boulder Dam cements* also discloses for the first time quantitatively the strength and heat producing properties of the various constituents of cements. Tri-calcium aluminate, which hydrates to a large extent in a single day, liberates more heat per unit of weight than any other major compound, and is shown to produce its principal strength in the first day or two of hardening, thereafter dropping off rapidly, causing materially lower ultimate strength.

Tri-calcium silicate, on the other hand, is shown to produce its strength during the secondary stage of hardening between the first and seventh day, with relatively little gain thereafter, evolving less heat than the aluminate. Cements high in dicalcium silicate show low early strength but a substantial increase in strength at later ages, so long as water is available for hydration. Having a correspondingly low and slow heat production such a cement suited to mass concrete work. It would seem from this, then, that two useful commercial cements are possible of production, the low-heat, slow setting cement for mass work, and a high heat, rapid setting cement for thin-wall structures and for cold weather concreting, blending these two for intermediate requirements.

Other constituents and physical characteristics contribute their part in a less important way as is well set forth in Professor Davis' report.

*Cem. Invest. for Boulder Dam. R. E. Davis, University of California. Journal, Amer. Concrete Inst., May-June 1934; *Proceedings*, Vol. 30, p. 485-497.

As further determined by the Boulder Dam research, the water-cement ratio to produce a constant consistency of concrete is affected by both chemical composition and the fineness of grinding of cement. Within certain limits, the finer the grinding, the less is the water requirement. The higher the tri-calcium aluminate the greater the water requirement.

AMOUNT AND EFFECT OF HEAT GENERATED BY CEMENT HYDRATION

In cold weather concreting, the question is raised "what amounts of heat are generated by the various cements in setting, which would aid heated water, heated aggregate form insulation, or all in bringing about a safe early strength?" Data herein submitted shed light on some aspects of this question, but not enough. The subject of heat generation by the hydration of cement, the effect of form insulation or other methods of assuring substantial strengths in winter concreting is a separate one on which some work has been done but much more is needed. The following may be said at this time: The amount of heat liberated by the hydration of portland cement affects the quality of the concrete only when the temperature of the concrete is raised or maintained constant thereby. The time-temperature relations of concrete in place will depend on:

(1) Total heat liberated by hydration of the cement and the rate of heat liberation. These depend somewhat on initial temperature of the mix.

(2) Specific heat of concrete, in the wet and dry state.

(3) Temperature gradient, conductivity and diffusivity of concrete.

(4) Surface-volume relations of the concrete mass.

(5) Insulation of surfaces.

(6) Temperature and velocity of the surrounding medium.

(7) Radiation effects—i.e., loss of heat at night and gain during the day, the gain and loss being largely affected by the color and texture of the surface.

For ordinary construction in which the rate of heat loss is relatively high, the rate of heat liberation may be of more importance in determining the maximum temperature rise and the mean temperature during the early life of the concrete than the total quantity of heat liberated. Knowing the appropriate constants for the various factors outlined, it should be possible to calculate the approximate time-temperature relation for any given condition of placement.*

While many data have been collected on the time-temperature relations for concrete in structures, except for researches for the Boulder

*See References 7, 8.

Dam there is little information on the amount of heat liberated or the rate of its liberation. Such information is necessary for calculation of the approximate temperatures of concrete or to correctly establish the thermal properties of the various cements now on the market. Due to the many variables involved, it now seems unlikely that it will ever be possible to calculate accurately the temperature at any given time in concrete on the job. However, such calculations would permit reasonable answers to such a question as "What initial temperature is necessary in order that the temperature of the concrete placed under given conditions will not be less than a required minimum." Temperature readings alone merely show the result of the specific conditions existing during their measurement, and are not applicable to other conditions except in a general way* Significant also are the effects of puzzolanic and accelerating materials on heat generation, of antifreezing materials studied by the Committee but not included in the present report.

PLACING CONCRETE BY VIBRATION

The use of high-frequency vibration in placing concrete has been generally approved since this committee began work on its report. The recent report of the Committee 609† makes detailed discussion here unnecessary but this development has an important bearing on job strengths at early ages.

HIGHWAY CONCRETE

In the highway field, as in building construction, a mass of data has been accumulated relative to laboratory tests, field tests and core tests, yet there seem to be no data by means of which the laboratory tests can be translated into figures representative of the road slab. Many investigations have been made of the merits of various curing media, slab designs and component materials, which within the one series of tests has given with reasonable accuracy a fair comparison of conditions as they occured then and there.

Perhaps the one series was on a sandy or gravelly subgrade which would absorb all the excess water squeezed out of the concrete by the finishing machine. Perhaps this test was conducted also in sunny, warm weather in which despite attempts to reduce the evaporation by the use of tarpaulins, the water-ratio of the dense, compacted concrete slab was lowered still further. Such a combination probably gave the optimum in slab strength as shown by cores, provided ample

*See References 1, 2, 5, 6, 7, 8, 25, 26, 27, 29.

†"Recommendations for Placing Concrete by Vibration," JOURNAL, Amer. Concrete Inst., March-April 1936; *Proceedings*, Vol. 32, p. 445.

curing moisture was available during the early period and provided also that high slab temperatures were not brought about by curing methods, yet the field-cured cylinders did not reflect those conditions. They received no compaction by the finisher, nor any water-cement ratio reduction by combined subgrade and weather action; on the contrary, unless carefully watched, due to their relatively small size they were easily dried out, thus stopping in whole or part the usual setting action, while any quick change in temperature would affect them materially as contrasted to the more gradual change taking place in the slab mass.

Field-cured cylinders are sometimes shipped to the laboratory considerably ahead of the test period, and during this shipment and storage, become thoroughly dried out. They are frequently tested in this condition, instead of being subjected to the standard preliminary water immersion. The test report, therefore, represents completely dry or partly moist concrete, with its correspondingly different strength result. Is it any wonder, then that the field cylinders vary distressingly from the laboratory-cured curve, or from the core test curves, such as they are?

Perhaps further along on the same project, the subgrade changes to adobe or tight clay, and the pavement is laid down in cloudy cool weather with rain at intervals. Little absorption or evaporation occurs; the concrete stays pretty much "as is" except for compaction by the finisher, while the rate of set is reduced by the lower temperature of the slab. That there is a difference in character of concrete is certain. Core tests would show it. Can not data be obtained as to the effect of temperature, of the compaction by the finisher, of subgrade conditions, etc., so that, given the quality material, laboratory cured, a subgrade soil index and the weather record, the approximate job-cured character can be computed? It would simplify matters materially, take more of the guess out of the fixing of the curing period, protect the weaker areas and in many cases permit the road to be opened sooner. F. C. Lang* writes:

> On account of the uncorrelated nature of the test data on strength available from different sources, it has only been possible to make comparisons of a general nature. For instance, the amount of moisture in the test specimens at the time of test was generally an uncontrolled variable, although it is now well-known that this factor can cause variations in strength which may equal those caused by differences in curing.

With reference to comparison of laboratory and field investigations, Mr. Lang continues;

> In general, tests made under controlled laboratory conditions should afford a basis for comparison of curing methods. However, the data studied show in some cases

*1930 Progress Report of Curing Committee, Highway Research Board.

diverging relations between laboratory and field tests, the causes for which must be determined before the final comparisons can be made.

For some curing methods these data show that the ratios of the strength of laboratory specimens to the strengths of laboratory specimens cured moist, decrease with age; while the ratio of field specimens cured by the same methods to field specimens cured by moist earth increase with age. These ratios are also higher for the field tests than for the laboratory tests. These curing methods were sodium silicate, integral calcium chloride, and minimum curing. Calcium chloride surface application showed a similar trend but of less magnitude. The methods of curing that tend to increase or conserve the internal moisture appear to maintain constant strength ratios to moist curing, in laboratory tests at all ages.

A possible explanation of these conditions may be found in the fact that in the later ages the laboratory specimens may have reached such a degree of dryness that hydration of cement has practically ceased while in the field there are various sources from which the water may be drawn. Such sources are rain, high humidity and subgrade. The survey of field tests data shows that these natural curing conditions are often present and exert a beneficial effect.

It should be possible through further research to define the relation between strength and retention of mixing water, so that specifications can be based thereon. Such a specification would furnish a valuable basis for measuring the value of proposed curing methods. To correlate such a specification with field conditions will require research on the influence of absorptive or semi-absorptive subgrades.

In view of the lack of such correlation data current investigations should require continuous graphical temperature records of the air, the slab, and subgrade; humidity records, tabulation of the character of the soil and weather records.

An excellent work done by the Missouri State Highway Dept. in hot weather involved much detail, included the various items outlined above, and followed the outline laid down by the Committee on Curing of the Highway Research Board. It apparently lacked, however, the connecting link between field and laboratory, the standard laboratory-cured cylinder for quality test.

The results of the various tests in this series, however, were studied with the idea of determining the relative effects of the various types of curing on the following properties and behavior of the pavement slab: (1) Temperature, (2) Moisture content, (3) Strength, (4) Linear expansion and contraction, (5) warping or curling action, (6) visible differences in the character of surface, and the spacing of transverse cracks. For the purpose of making comparisons of the several methods of curing, based on their influence on the above, an attempt was made to group the data in a manner which would tend to balance the effects of uncontrolled variables such as weather conditions, subgrade characteristics, and qualities inherent in the concrete at the time the curing process was begun. Temperature records obtained showed that, regardless of the method of curing, the time-temperature curve for the

concrete at the surface of the pavement followed the general contour of the corresponding air temperature curve. The bottom of the slab had a considerably smaller range, while the maximum and minimum temperatures at the bottom lagged from two to three hours behind those of the top.

Although the temperature of the concrete depended primarily upon the temperature of the air, numerous measurements made simultaneously of the temperature of the concrete in sections of the pavement cured by different methods showed a difference of as much as 40°F. That the methods of curing were responsible for this difference, there is little doubt. Dark-surfaced curing in warm sunny weather produced a slab temperature 40° above that of water-curing, reaching temperatures as great as 140°. This high temperature had the disadvantage of producing much greater curling or warping, greater volume change, and a greater amount of cracking. This type of curing produced not only the highest pavement temperature but also the greatest difference in temperature. The same material, however, given a white surface by a spray or dusting of hydrated lime reflected instead of absorbed the heat and apparently produced a curing condition more like water insofar as the effects of temperature are concerned. Kraft paper curing, perhaps somewhat unsatisfactorily handled, showed a temperature effect a little less than midway between ponding and black curing, fairly uniform in variation, while minimum 24 hr. burlap, calcium chloride surface application and sodium silicate developed temperature variations a little greater. It is evident, therefore, that the black surface might be more suitable for winter work, aiding by heat absorption the maintenance of higher pavement curing temperatures, during the period of lower air temperature; while the light colored surfaces should be specified for summer work where economically suitable.

For beam and core tests, specimens of beams for field-curing were made in water-tight molds, one set being exposed to air after 24 hr. of curing under wet burlap, while the other set was buried in damp sand, both intended to serve as control tests on the concrete being placed. Core tests were made to obtain a direct measure of the strength of the concrete in the pavement.

The beam tests failed utterly to perform their allotted part. There should have been a high correlation between the beam and core strengths of each section, all things being equal, but the correlation, if any existed, was very low. The reason for this is perhaps not hard to find. Being made in watertight molds of shallow section as compared with the pavement, they lost the absorptive effect of the subgrade, the

compacting effect of the finishing machine, and the effect of volume as related to exposed surface area, while both temperature and moist curing conditions were different.

Core tests varied due to variations in the inherent potential strengths of the concrete, to the degree of compression which the concrete received when being placed in the pavement, to the amount of moisture absorbed by the subgrade or removed by manipulation during the plastic stages, to temperature during the early curing period, and to differences in the natural curing conditions to which the pavement sections were subjected. The question is raised as to the uniformity of pavement concrete from top to bottom—whether the stiff mixes become thoroughly and uniformly compacted by the finishing machine, or whether there is not a variation which affects the core test adversely. The introduction of internal vibration into the placing and finishing of the slab may produce the answer.

In the Missouri tests, standard practice was adhered to constantly, including the frequently-neglected preliminary immersion of the test specimens in 70° water. In field beam tests generally elsewhere, this is believed to be the exception rather than the rule; the tests usually being made at the roadside on specimens dry or partly dry, at temperatures varying considerably from the standard, yet wonder is expressed why these results do not correspond with core tests or why the similarly cured field cylinders shipped to the laboratory and dry-cured there until the time of test cannot be correlated with other tests. Such loose practices must be eliminated if our money spent for testing is to produce usable results.

As shown by the Texas work, (page 1) the modulus of rupture of 1:2 mortar beams tested at 14, 28 and 100 days at varying temperatures, compared with standard 70° testing, varied uniformly from approximately 65 per cent at 140°F. to 110 per cent at 32°F., all test pieces having been cured at 70° and brought to the temperature of test immediately prior to breaking. This series of tests, though not at all complete, shows clearly the effect on strength of the temperature just prior to and at the time of test.

It is evident that in all tests the temperature at the time of test should be closely controlled, since the results are of value only when they can be compared with standards. Establishment of these standards for minimum strength should require careful adherence to the temperature taken as normal, since any appreciable variation from it will detract from the value of the test. In this respect, as in others, it is important that this work be done by or under the close direction of laboratory-trained men thoroughly up to date with respect to

current research, yet whose background has included such work, who are alive to the difficulties and characteristics of construction practice, and who have an appreciation of the value of coordinating research with practice.

CONTROLLING STRUCTURAL CONCRETE BY CORE TEST

As this report is being given final preparation, recently-developed equipment,* simple of operation, portable and of moderate cost, is being used on one of the author's projects, to cut cores from the structure for test comparison with laboratory-cured standard cylinders. Its ease of handling, coupled with coring costs materially less than those of previously-available equipment is opening a new field of concrete control—by taking cores from the finished structure. That this means much for job concrete quality, placement and curing is obvious. Instead of the various forms of cheating resorted to, to secure high-test quality cylinders, the knowledge that cores will be taken at random from the structure with penalties for low performance, will cause constant care to be taken to insure adequate results. It will soon be realized that longer curing will produce better concrete in the structure, and that poor placing methods will result in low-test cores. Consequently building concrete will be given the same care in design, placing and curing that highway concrete has secured under similar regulation.

Core test control has been used on a number of buildings in Southern California, and is increasing in popularity as its many advantages are realized.

SUMMARY

Subject to corroboration by further tests, the following statements may be made at this time:

1. Under careful control, concrete in the structure can be generally of higher strength and of greater density than that indicated by test cylinders laboratory-cured. Due to the lack of adequate curing, together with rule-of-thumb design and careless placing methods, however, concrete in the ordinary job should normally be expected to have somewhat lower comparative characteristics.

2. Concrete in monolithically-finished slabs is of higher strength and greater density than the same concrete placed in walls. This difference is greatest when the concrete is hand-placed, of a somewhat fluid character. Low-slump concrete placed by vibration shows a smaller difference though of much higher strength generally, due, apparently, to the smaller amount of free water in the mix to be

*Developed by Frank L. Howard, formerly with the Smith Emery Co., Testing Engineers, Los Angeles.

absorbed by forms, lost by leakage or otherwise removed by evaporation, dry-top monolithic slab finish, etc.

3. Concrete in walls and columns has its greatest strength and density at the lower part of the section placed at one time, decreasing toward the top of the section due to water-gain. This again is much less when using low-slump, vibrated concrete.

4. Longer curing than the usual 3 to 7-day period, coupled with properly designed, leaner concrete, tends to produce a more economical job, of equal job strength to wetter concrete of the same water-cement ratio, yet having lower shrinkage and expansion characteristics.

5. Test cylinders, job cured, are not usually representative of the concrete in the structure, due to unavoidable differences in moisture and temperature curing conditions as well as in the water-cement ratio at time of set.

6. In all strength tests of portland cement, temperature at the time of test should be closely controlled, since results are of value only when they can be compared with standards. Test temperatures higher than 70° decrease the strength materially, while lower temperatures increase the strength to a smaller degree.

7. Adequate curing requires the continued presence of moisture supplied by repeated wetting of all parts of the structure, the necessary frequency of which is dependent upon the air temperature and upon the character of the constituents of the cement used.

8. Concrete controlled by cores cut from the finished structure is now economically possible and advisable due to recent perfection and simplification of portable core-cutting equipment.

REFERENCES

1. B. S. A. (*Building Science Abstracts.* Compiled by Building Research Station, Great Britain.) Abstract No. 75, 1928. On the development of heat during the setting of cement. A. Frey Teknisk Tidskrift, 1927, Vol. 57, No. 12, p. 23.

2. B. S. A. No. 247, 1928. Heat Changes During Setting of Alumina and Portland Cements. A. A. Jakkula, *Eng. News Record*, 1927, Vol. 99, No. 24, p. 955-6.

3. B. S. A. No. 630, 1928. Thermal Analysis of Cement-Admixtures and Mixed Cements. Shichire Uchida Sendai Higher Technical School Research Report, 1926, Vol. 5, No. 2, p. 264-82, (*Ceramic Abstracts*, 1928 7 (3) 138.

4. B. S. A. No. 624, 1928. Temperature rise during the setting of portland cement containing calcium chloride. Shichire Uchida Sendai Higher Technical School Research Report, 1926 5 (2) 322-9, (*Ceramic Abstracts*, 1928 7 (3) 141.

5. B. S. A. No. 889, 1929. Heat of reaction of Portland Cement and Mortar. Anon. *Pit and Quarry*, 1929, 17 (8) 93, (*Amer. Ceramic Soc. Abstracts*, 1929, 6 (4) 247.

6. B. S. A. No. 2308, 1929. The use of cement developing heat for concrete work in frosty weather. J. Grantholm: *Byggm-staren*, 1929, 10 (32) 10.

7. A. C. I. Abstract, 1931, p. 193. Report on the Swedish investigation and ex-

periences concerning shrinkage and temperature in concrete. M. Royen, Summary Reports 1st International Congress for Concrete and Reinforced Concrete, Liege, 1930

8. A. C. I. Abstract, 1931, p. 291. Calculation of heat reactions in winter concreting. Max Mayer, *Zement* (Germany), April 2, 1931, 20 (14) 322-5. Reference is made in the paper to the "Gefrierschacht—Untersuchung von, Dr. Grün (*Zement*, 1928, No. 37, und ff.). (This article does not appear to be abstracted by A. C. I. or B. S. A.)

9. A. C. I. Abstract, 1931, p. 292. Differentiation of the properties of cements. Stig Giertz-Hedstroom and Donovan Werner. *Zement* (Germany), March 5, 1931, V. 20, No. 10, p. 213-5.

B. S. A. No. 411, 1930, p. 66. Autumn risks in concrete construction. F. Emperger. *Beton u. Eisen*, 1929, 28 (24), 448-50.

11. B. S. A. No. 975, 1930, p. 167. The effect of cold, damp weather on the high, early strength of rapid-hardening concrete with various proportions of water and cement. Orthaus. *Beton u. Eisen*, 1930, 29 (6), 113-9.

12. B. S. A. No. 89, 1931, p. 21. Some properties of high-alumina cements from six countries. P. H. Bates. First Communications of the New International Assn. for the Testing of Materials. Group B, 211-8.

13. B. S. A. No. 1000, 1931, p. 195. Temperature of maturing of concrete with rapid-hardening cement. N. Davey. *Concrete and Constructional Engineering*, 1931, 26 (5) 311-5.

14. B. S. A. No. 1001, 1931, p. 196. Effect of heating materials and applicances on the rate of hardening of rapid-hardening portland cement. N. Davey. *Concrete and Constructional Engineering*, 1931, 26 (6) 359-64.

15. B. S. A. No. 1002, 1931, p. 196. The sensitivity of concrete-making materials to heating in winter construction. L. David. *Zement*, 1931, 20 (10) 118-21.

16. B. S. A. No. 1384, p. 269. Measurement of the temperatures in concrete during hardening. F. G. Perez. *Ingenieria*, 1930, 4 (11) 418-20, 446.

17. B. S. A. No. 1386, 1931, p. 270. The effect of low temperatures on the setting and hardening of concrete. A. Pogany. *Zement*, 1931, 20 (20) 478-80.

18. B. S. A., No. 1827, 1931, p. 338. The effects of temperature on the mechanical properties of concrete. E. Marcotte. *La Technique des Travaux*, 1931 (6) 389-94. *Bauing*, 1931 12, *Literaturschau*, 1.

19. B. S. A. No. 2052, 1931, p. 371. The new works of the Rodauner Zementfabrik A. G. F. Bohm, *Beton u. Eisen*, 1931, 30 (16) 289-93.

20. B. S. A. No. 2054, 1931, p. 371. Some observations on the modulus of rupture of frozen concrete beams. A. P. Anderson. *Public Roads*, 1931, 12 (7) 195-6.

21. B. S. A. No. 2055, 1931, p. 372. Temperature rise of Concrete. N. Davey. *Concrete and Constructional Engineering*, 1931, 26 (10) 572-5.

22. A. C. I. Abstract, 1931, p. 12. Report on experiments to study cement mortar strength a few hours after mixing. Alex. Hasch. *Zeitschrift Ostr. Ing. Arch. Vereins* (Austria), April 17, 1931, Vol. 83, No. 15-16, p. 141-2.

23. A. C. I. Abstract, 1931, p. 32. Effect of low temperatures on setting and hardening of concrete. Adalbert Pogany. *Zement*, May 14, 1931, Vol. 20, No. 20, p. 478-80.

24. A. C. I. Abstract, 1931, 0. 35. Concreting in freezing weather. D. A. Plotnikov, P. B. Tarassov, Y. A. Yochelson and I. F. Yastrebov, Bul. 5, Institute for Scientific Concrete Research, Leningrad, U. S. S. R., 1931.

25. Hoover Dam Cement Specifications Tentatively Formulated. J. L. Savage. *Eng. News Rec.*, November 10, 1932.

26. Development of Low-heat Cement for Mass Concrete. R. W. Carlson. *Eng. News Rec.*, October 20, 1932, p. 461.

27. Heat Evolved by Cement in Hardening. Wood, Steinour and Starke. *Eng. News Rec.*, October 6, 1932, p. 404.

28. Special Low-heat Cement for Pine Canyon Dam. *Eng. News Rec.*, October 6, 1932, p. 410.

29. Heat Developed by Cement while Setting and Hardening. F. L. Meyers. Western Portland Cement Co., El Paso, Texas.

30. Properties of Mass Concrete. Report of Committee 108. R. E. Davis and C. E. Troxell. *Proceedings*, Amer. Concrete Inst.; *Proceedings*, A. C. I., Vol. 29, 1931, p. 305-418.

31. Temperatures Developed in Mass Concrete and their Effect upon the Compressive Strength. R. E. Davis and G. E. Troxell, *Proceedings*, A. S. T. M., Vol. 31, 1931, Part II, p. 576-94.

32. Variations in Standard Portland Cements. Report of Committee 202. P. H. Bates. *Proceedings*, Amer. Concrete Inst., Vol. 26, 1931, p. 65-95.

For such discussion of this report as may develop readers are referred to the JOURNAL *for January-February 1937. Discussion should reach the Institute Secretary by December 1, 1936.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY, 1937

Discussion of the Report of Committee 107:

Properties of Job-Cured Concrete at Early Ages*

H. H. SCOFIELD†

In response to the committee's request for more data on the relation of job-cured concrete to the 70° cured standard control cylinders, the following is here submitted:

These results were obtained during inspection and control of concrete on three multiple-story reinforced concrete buildings near Ithaca.

No attempt has been made to separate the different classes of concrete. Column concrete, slab concrete, footings and wall concrete are all averaged together.

Standard practice (A. S. T. M. C 31-33 and C 39-33) was used in making and testing the specimens. The job-cured specimens were kept under the same conditions as the concrete in place. Protection from freezing temperatures was given to both. In making the test specimens, just enough concrete was sampled to mold a job cylinder and a laboratory control cylinder. The molds were the usual waxed paper molds. All cylinders were capped on the job. The laboratory control specimens were removed to the moist room within 48 hours. The temperature of the moist room ranged from 68°F. to about 73°F.

The control specimens were tested moist. If job specimens were brought to the laboratory a little before time of test they were kept outdoors. All job specimens were tested in the same condition as they arrived at the laboratory.

It is the intent, of course, that the laboratory control specimens should give the quality of concrete being furnished to the job, the

*JOURNAL, Amer. Concrete Inst., Sept.-Oct. 1936; p. 41.
†Prof. of materials, Cornell University, Ithaca, N. Y.

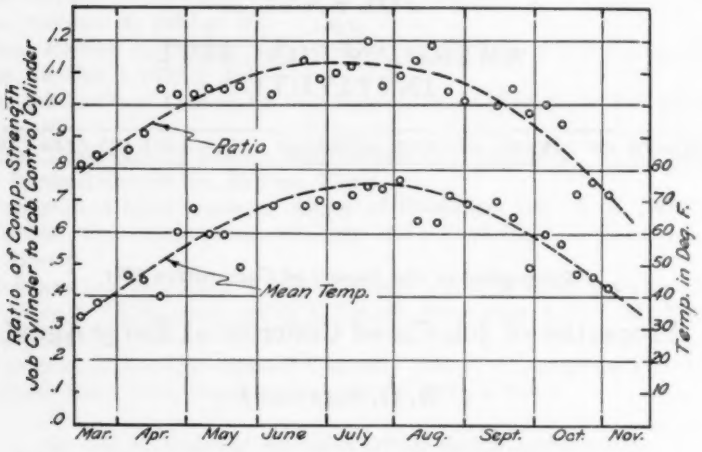


FIG. 3—COMPARISON OF COMPRESSIVE STRENGTH AT 28 DAYS—6 x 12-IN. JOB CYLINDERS AND CONTROL CYLINDERS

COMPRESSION TESTS OF CONCRETE—A COMPARISON OF JOB AND CONTROL CYLINDERS

Week Beginning	Temp. Deg. F.			No. of Cyl. Each Kind	Comp. Strength p.s.i.		
	Max.	Min.	Mean		Job A	Control B	Ratio A/B
Mar. 16	49	15	33.8	3	3018	3705	.81
Mar. 23	51	31	37.7	13	3359	3975	.84
Apr. 6	54	36	46.3	11	3469	4024	.86
Apr. 13	59	35	45.3	6	3623	3997	.91
Apr. 20	49	34	39.6	8	3757	3557	1.05
Apr. 27	75	51	60.9	8	3187	3112	1.03
May 4	78	44	67.2	8	2797	2723	1.03
May 11	70	49	59.2	4	2765	2630	1.05
May 18	73	43	59.2	8	2845	2718	1.05
May 25	55	40	49.0	4	3275	3097	1.06
June. 8	84	64	68.2	8	2513	2450	1.03
June 22	87	59	68.2	5	2566	2238	1.14
June 29	78	56	70.0	7	2871	2667	1.08
July 6	78	57	68.5	5	2690	2450	1.10
July 13	81	57	71.4	11	3299	2955	1.12
July 20	82	56	74.7	13	3660	3054	1.20
July 27	85	57	73.3	4	3257	3097	1.06
Aug. 3	97	67	76.6	4	3770	3460	1.09
Aug. 10	75	47	63.6	2	3300	2895	1.14
Aug. 17	77	53	63.6	11	3016	2525	1.19
Aug. 24	83	57	71.5	14	3315	3200	1.04
Aug. 31	73	50	69.0	1	3740	3700	1.01
Sept. 14	81	55	69.5	12	3301	3315	1.00
Sept. 21	81	58	64.6	2	2190	2085	1.05
Sept. 28	52	37	48.5	7	3110	3246	.97
Oct. 5	64	49	59.4	4	3513	3532	1.00
Oct. 12	68	46	56.5	4	2593	2763	.94
Oct. 19	40	32	47.0	4	2005	2790	.72
Oct. 27	46	31	46.2	5	2172	2845	.76
Nov. 2	41	27	42.7	4	2227	3095	.72

Notes:
1. The maximum temperature given is the average of the maximums of temperature on days, cylinders were molded.
2. The minimum temperature given is the average of the minimums of temperatures on the day after cylinders were molded.
3. The mean temperature given is the mean for the entire week during which specimens were molded.

measuring stick being a previously determined or stipulated value of strength at 28 days, of a standard specimen cured moist at a temperature approximately 70°F. Just how close these control specimens come to such a pre-determined or stipulated value depends upon the control of the proportioning and mixing operations on the job. With close control, a reasonable agreement should prevail.

It should be evident, however, that the job cylinders will be subjected to varying temperatures and possibly also to considerable variation in humidity. This means that in the cooler months the ratio of strength of job cylinder to control cylinders may be considerably less than unity, and that in such periods of the year the specimens will take a longer curing period to attain the same strength as the control cylinders. They should, however, attain the same strength as the control cylinders at a later date provided they are not permanently injured by freezing or by drying out.

In the test results here reported, the job cylinders are below the control cylinders in strength during March and most of April. They are above the control cylinder strengths during the warm months and those made in October and November are again below the control cylinder strengths.

It is inevitable that the question should arise as to the comparison between both of these types of separately molded specimens and the concrete actually in place on the job. The answer, of course, depends upon a number of practical conditions. Good concrete furnished to the work may suffer a great deal by improper methods of placing and curing. Assuming fair conditions of placing and curing, it is the belief of the writer that the job cylinder results represent about the minimum to be expected for the concrete in place, because mass temperatures are higher and mass drying out is less than in the smaller test specimen. Numerous tests have shown this to be true.

CLOSURE BY CHAIRMAN HARLAN H. EDWARDS

In closing discussion of the report of the Committee 107, I should like to bring the report down to date by putting into the record data just made available,* concerning cores recently cut from several representative buildings in Southern California in course of construction.

As mentioned by Professor Scofield in the last paragraph of his discussion, "Good concrete furnished to the work may suffer a great deal by improper methods of placing and curing." This is undeniably true on many jobs. Where means exist of definitely determining the con-

*Courtesy, Concore, Inc., Los Angeles, Calif.

FIG. 4—CORES NO. 1 - 4

crete characteristics such as by cutting cores, assumptions *must not be made* and *are not being made* on projects of the Los Anglees Board of Education, as well as on those of several prominent structural engineers and architects in the Southern California area.

In the determination of the job characteristics of concrete on one of the high schools under construction in Los Angeles, cylinder records are compared with core records in Table 1. Cores were cut as nearly as possible from the locations represented by the cylinders. Core No. 1 was cut across a cold joint to demonstrate the bond between sections of concrete placed at different times and to determine the strength of the concrete at the top of the previously placed wall, and is, therefore, the lowest of the group. Likewise, the variation between low and high in each group is considerably greater in cylinders than in cores. While these parallel tests are not made of concrete of equal ages, the cores demonstrate conclusively that the finished building contains concrete of specified strength. Fig. 4 shows cores No. 1 to 4 and demonstrates clearly the value of cores to

TABLE 1—CYLINDER STRENGTHS AND CORE STRENGTHS

(Representing concrete with same approximate locations—28-day results)

	Cylinders				Cores	
	p.s.i.		p.s.i.			p.s.i.
1	*2360	—	2567	Average—2463	1	2312—Low
2	*1972—Low			" —1972	2	2870
3	2435	—	2431	" —2433	3	4014
4	2923			" —2923	4	3704
5	3419	—	3025	" —3222	5	3567
6	3025	—	2923	" —2973	6	4093—High
7	3246			" —3246	7	3421
8	2839	—	4394—High	" —3616	8	3154

Average—of all cylinders—2856 p.s.i.; of all cores—3517 p.s.i.
Variation in cylinders between the lowest and highest strengths of 2422 p.s.i.
Variation in cores between the lowest and highest strengths of 1781 p.s.i.
*Indicates that the 7-day test on the twin cylinder was below requirements.

the engineer in visually determining the characteristics of the concrete in place. Core No. 2, it will be seen, is of porous concrete and, as would be expected, has a lower crushing strength. Core No. 3 exhibits a more dense structure and has a much higher strength.

TABLE 2

	Concrete Cores Average 30 Days	Concrete Cylinders 7 Days	Concrete Cylinders 28 Days
School			
A — Minimum	2889		1632
Maximum	4800		3680
Average	3433.4		2897.2
B — Minimum	2272		2112
Maximum	4156		3638
Average	3317.6		2769.1
C — Minimum	2151		2239
Maximum	4330		3944
Average	3400.5		3395.2
D — Minimum	2860		1597
Maximum	3158		2746
Average	2994.3		2171.5
E — Minimum	3807	GUNITE	5483
Maximum	4099	(only cyl. made so far)	
Average	3953		

Since this work was done, the cutting of cores has been specified in place of the 28-day cylinders on a number of the schools. Briefly, the results are shown by Table 2. It will be noted that a number of the 28-day cylinders were below the permissible minimum, particularly on Schools A & D. Under the specifications this would require a load test on each section covered by the cylinder. This expense was avoided by the results of the core tests, which showed the concrete to be of specified strength. Difficulties in placing, variations of transit mixed concrete from batch to batch, certain runs of concrete made much wetter than normal and not well placed, as well as inexperience of the inspector in making cylinders, are also brought out in these different tests and account for the variations shown in the tests. The tests on School E were of gunite used on a reconstruction job and show the apparently characteristic difference between the strength of gunite as shown by the cylinder and the actual strength existing in the structure. This difference in strength is accounted for by the fact that in shooting the cylinder with gunite, a much greater rebound can be made to occur by the operator, thereby giving a gunite in the cylinder having a mix of approximately 1:2.5 or 1:3, while the mix of the gunite actually shot in the wall is nearer 1:4, resulting from a 1:4.5 original mix. The core also shows the laminations of the gunite, due to improper placing, which are of detriment to the structure.

TABLE 3

Cores Location	Compressive Strength, p.s.i., Corrected to H/D			
	East Addition		West Addition	
	Column	Beam	Column	Beam
Basement	4458	5461	4363	3896
First Floor	3889	3899	4172	4169
Second Floor	4403	3503	4247	4784
Third Floor	3634	3906	4082	4155
Fourth Floor	3628	4411	4902	3908
Fifth Floor	4243	3831	3519	3699
Sixth floor	4518	4354	4127	3862
Average	4110	4195	4202	4068
Loading platform	4895		4741	

Cores cut from columns: Minimum — 3519 (31 days old)
Maximum — 4518 (30 days old)
Average — 4156
Cores cut from beams: Minimum — 3503 (31 days old)
Maximum — 5461 (52 days old)
Average — 4131
Average of all cores cut — 4231 pounds per square inch.

Concrete Cylinders:

The results from 54 28-day cylinders taken on this job were not segregated as to columns and beams. However, the results as a whole are as follows:

Minimum — 2662
Maximum — 4405
Average of all concrete cylinders — 3483

According to the specifications, any strength which fell below 3000 lbs. would require a load test. This expense was eliminated due to the fact that cores were cut. The lowest core cut showed a strength of 3519 pounds, which is far in excess of the specification requirement. There was a total of six concrete cylinders having a sternght lower than 3000 lb.

The Sears, Roebuck & Co. buildings, designed and controlled structurally by Oliver G. Bowen, Structural Engineer, are an excellent example of the use of cores in control of the work. These buildings were the first in Los Angeles to be designed for 3000 lb. concrete, approved by the Department of Building and Safety, and as such the results are especially significant. They are given in Table 3. Since this was the first work of its kind, concrete cylinders were also taken (though not generally necessary) to demonstrate the strength as usually obtained. The minimum cylinder tests were of the concrete on the third floor, east addition, and under the specifications, a test below the permissible would have required a load test, which would in this case have cost from five to seven hundred dollars.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . SEPTEMBER-OCTOBER 1936

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviews.

Clinker cooling

F. LIPINSKI, *Tonindustrie Zeitung*, Vol. 60, No. 26 and 27, 333-6, 355-6, March 30, Apr. 2, 1936.
Reviewed by A. E. BEITLICH

A historical review of various methods and patents on clinker cooling. Special consideration is given to heat, economical questions and the effect of cooling on the quality of portland cement clinker.

Disadvantages of our present cement specifications

WALTER DYCKERHOFF, *Zement*, Vol 25, No. 22, May 28, 1936, pp. 367-374.
Reviewed by L. T. BROWNMILLER

A review in which the author points out that present cement test specimens do not duplicate the actual concrete specimens because of the use of standard sand, different conditions of filling forms, etc.

Solved and unsolved problems of clinker investigations

HANS KUHL, *Zement*, Vol. 25, No. 20, May 14, 1936, pp. 335-338 and Vol. 25, No. 21, May 21, 1936, pp. 351-354. Reviewed by L. T. BROWNMILLER

Some of the unsolved problems of cement chemistry are those of the status of liquid formation on burning, the disposition of the liquid on cooling, the lime saturation point of cement mixtures, the effect of the minor constituents and the addition of fluxes to cement mixtures.

Determination of the mixing proportions and cement content of cement mortar and concrete

H. W. GONNELL, *Zement*, Vol. 25, No. 24, June 11, 1936, pp. 411-413 and No. 25, June 18, 1936, pp. 426-429. Reviewed by L. T. BROWNMILLER

The method for determining the mixing proportions of concrete is described as it is used by the State Testing Materials Laboratory in Berlin-Dahlem. The method depends largely on HCl—solubility and therefore is subject to the usual limitations.

Barium carbonate as protective agent for concrete against sulfate waters

HANS JORDT, *Tonindustrie Zeitung*, Vol. 60, No. 35, 443-4, Apr. 30, 1936.
Reviewed by A. E. BEITLICH

Laboratory experiments showed that concrete containing admixtures of ½ to 1 per cent barium carbonate (witherite) was more resistant to sulfate waters than

concrete which did not contain this admixture. The barium carbonate forms insoluble barium sulfate with the sulfates present in solution and thus prevents the formation of destructive double salts with the calcium oxide of the cement. The barium sulfate furthermore forms a protective coating.

Void calculator for aggregates

C. E. HEINZ. *Rock Products*, Vol. XXXIX, No. 6, June 1936, p. 48. Reviewed by ROY N. YOUNG

A simple monographic chart is shown for the calculation of void space in aggregates. The chart is based on methods approved by the American Society for Testing Materials and the American Association of State Highway Officials. It involves the variables—void space, specific gravity of the mineral particles and weight per unit volume in terms of grams per .1 cu. ft. or lb. per cu. ft.

How to prepare concrete aggregates that please customers

STANLEY M. HANDS, *Rock Products*, Vol. XXXIX, No. 7, July 1936, pp. 38-42.
Reviewed by ROY N. YOUNG

The scope of this paper is broader than might be indicated by its title. The author has discussed rather thoroughly the factors involved in the relationships among preparation of aggregates, equipment for placing concrete and quality of concrete. Numerous photographic illustrations representing field conditions are used.

Temperatures in concrete arches

C. R. WHYTE (Engineering of Bridges, District of Columbia). *Engineering News-Record*, Vol. 116, No. 20, May 14, 1936, pp. 693-695. Reviewed by N. M. NEWMARK

Report is made of temperature readings taken at frequent intervals at various sections in the ribs of two concrete arches in Washington, D. C. The purpose of the observations was to determine the optimum time of pouring the key sections. In general, maximum temperatures were reached in one to two days, and the effects of the initial rise in temperature appeared to have been dissipated in about 10 days.

Precasting record-size pipe

Engineering News-Record, Vol. 116, No. 24, June 11, 1936, pp. 852-854.
Reviewed by N. M. NEWMARK

Precast concrete pipe with inside diameter of 152 in. weighing 43 tons per 12-ft. length and designed for a maximum internal pressure head of 80 ft. is being used for part of the main pipe line in the distribution system of the Colorado River aqueduct. This article describes details of the casting, curing, and handling operations by which sixteen sections are turned out daily.

New process for rapid hardening of concrete

Concrete Bldg. & Concrete Products, Vol. XI, No. 5, May, 1936, pp. 93-95, and in *Cement & Cement Manufacture*, Vol. IX, No. 5, May 1936, pp. 99-102. Reviewed by J. C. PEARSON

An abstract of Freyssinet's paper before a joint meeting of the Engineers of France and England last winter is given in this journal. A review of this paper was previously presented (see A. C. I. JOURNAL, May-June, 1936, p. 732) and is mentioned here only as accompanying a description of an English Patent (No. 431,484-1934) obtained by M. Freyssinet for a method of making concrete pipes, poles and other articles.

A double hammer crusher

FR. KORTE, *Zement*, Vol. 25, No. 20, May 14, 1936, pp. 339-341 Reviewed by L. T. BROWNMILLER

A description of a double hammer crusher which was brought on the market in 1933. The advantages claimed for this crusher are: (1) The eccentric shaft can be

kept smaller in diameter because the surface speed of the shaft in the bearing can be decreased, which results in power saving. (2) Because of decreased friction in the bearings savings can be effected in lubricants, replacements and repairs. (3) In none of the installations to date has there ever been overheating of the shaft bearings.

Measurements of the grain size of cement

Tonindustrie Zeitung, Vol. 60, No. 26, 341-2, March 30, 1936 Reviewed by A. E. BEITLICH

A description is given of a method developed by R. Tillmann to determine the grain sizes of cement by counting individual sizes under the microscope. The field of the sample is projected against a screen and the grain composition can be determined within 1½ to 2 hours with an accuracy of usually less than 2 per cent. The special microscope and projector are described and illustrated.

Concrete placing machines line 92 miles of tunnel

Engineering News-Record, Vol. 116, No. 25, June 18, 1936, pp. 876-879
Reviewed by N. M. NEWMARK

Concrete lining in the 92 miles and 17 tunnels of the Colorado River aqueduct is being placed exclusively by mechanical equipment. In addition to concrete pumps, four types of concrete "guns," which deliver intermittent batches of concrete with compressed air as a motive force, are used. In most cases mixers are mounted on the units operating in the tunnel but a few machines are served by mixers outside the tunnel portal. Up to June 1, 800,000 cu. yds. of concrete had been placed, or 47 per cent of the total. The technique of concrete placement is described in the article.

Concrete tunnel lining pumped down drilled holes

EMORY D. ROBERTS (Professor of Civil Engineering, Marquette University), *Engineering News-Record*, Vol. 116, No. 24, June 11, 1936, pp. 839-841. Reviewed by N. M. NEWMARK

Concrete is pumped from the surface into the forms for the lining of small bore interceptor sewer tunnels in Minneapolis. Three pumping units are fed by 1½-yd. transit mixers charged at the city's central batching plant. The concrete is mixed en route to the pumps over a maximum hauling distance of 5 miles. A 7-in. discharge line leads from the pump to the forms. Pumping is carried on up to distances of 800 ft. from the pump. Concrete with a slump of 2 to 3 in. is used and successfully packed into the forms by admission into the pipe of air at 110 lb. pressure at regular intervals. Details of the operation of the method are described.

Annual meeting of the Association of German Portland Cement Manufacturers

CZERNIN, *Tonindustrie Zeitung*, Vol. 60, No. 28-30, 363-5, 378-9, 390-3, Apr. 6, 9, 16, 1936.
Reviewed by A. E. BEITLICH

Detailed reviews of papers presented at the Berlin meeting, March 31 to April 1, 1936, including the following: Dyckerhoff: The faults of our present cement specifications; Graf: Selection of cements for concrete roads; Haegermann: Chemical composition of portland cement for the construction of concrete road slabs; Schwiete: Development of cements for concrete roads; Haegermann: Report on studies made at the Research Laboratory; Kruger: Report on studies made at the Government Institute for Testing Materials at Berlin-Dahlem; Eitel: Properties of portland cements high in magnesia; Nacken: The reactions of calcium silicates and aluminates in water solutions; Kuehl: Solved and unsolved questions in clinker research.

Thermochemical principles for the preparation of cements

H. ELSNER V. GRONOW, *Zement*, Vol. 25, No. 26, June 25, 1936, pp. 437-442 and No. 27, July 2, 1936, pp. 453-458. Reviewed by L. T. BROWNMILLER

Author measured (1) the specific heats of the raw materials, as well as those of the

pure clinker minerals and of portland cement clinker over the temperature range from 20° to 1500° C. The specific heats of cements were calculated from the values obtained for the clinker minerals.

(2) Experimental determinations were made (a) of the heat involved in the decarbonation of limestone at 907°; (b) of the endothermic effect during the dehydration of the clay minerals; (c) of the exothermic effect during the formation of portland cement clinker from clay and limestone and from slag and limestone; (d) of the heat necessary for the formation of melt at higher temperatures.

(3) The calculation of the complete heat requirements for burning of cement on the basis of the values obtained under (1) and (2).

The interaction of concrete slabs and steel beams

RUDOLF KOLM, *Betong* No. 1, pp. 1-19, 1936. Reviewed by INGE LYSE

An extensive study of the interaction between concrete slabs and steel beams was made on Swedish highway bridges. The strains were measured by Huggenberger tensometers and the loads were applied by means of heavy trucks. The results of eleven such load tests of new girder bridges with reinforced concrete floors are presented in this report. The test results are compared with the design computations, and it is deduced that the slabs in general act with the structural steel beam or girder within the experimental limits. However, the observations do not warrant this interaction to be counted on in all cases or give information as to how much interaction may be safely used in design. It is therefore recommended that further tests be made in order that definite design methods may be included in future code specifications.

Lime content and resistance of cement against chemical attack

VICTOR RODT, *Zement*, Vol. 25, No. 28, July 9, 1936, pp. 469-471. Reviewed by L. T. BROWNMILLER

On the basis of new experiments, the greater resistance claimed for slag cements against chemical attack seems to be due to their lower lime content. By lower lime content is not meant the total lime content of the cement but only the free $Ca(OH)_2$ of the hardened product. The experimental data include determinations of the free $Ca(OH)_2$ and combined water of the following types of cements after 1, 3, 6 and 19 months of water storage: portland cement *P*, super portland cement *S*, cement *A* with 75 per cent slag, cement *N* with 63 per cent slag, portland cement *P* with 29.6 per cent trass, super portland cement *S* with 29.6 per cent trass. At all ages cements *A* and *N* showed lower free lime and lower combined water contents than the other types of cements. No evidence is included, however, to show that cements *A* and *N* were more resistant to chemical attack.

The simultaneous production of iron and portland cement in the rotary kiln

PATRICIO PALOMAR COLLADO, *Tonindustrie Zeitung*, Vol. 60, No. 10, 11 and 22, 123-4, 141-3, 285-6, Feb. 3, 6 and March 16, 1936. Reviewed by A. E. BEITLICH

In this process, developed by Lucien Basset, the formation of carbon dioxide and water vapor in the burning process is held back, and the combustion is so directed that carbon monoxide and hydrogen are formed which results in a complete reduction of the iron ore. The corrosive action of the very liquid slag is decreased by the addition of calcareous materials to approach also the composition of portland cement clinker. The raw materials, consisting of finely ground iron ore, limestone and coal or coke (for reduction), are fed to the rotary kiln in the form of moist powder or slurry. In the interior of the kiln, the moisture is driven off first and the combustion of the coal begins with reduction of the iron ore. Then the limestone gives off the

carbon dioxide and finally is heated to the proper clinkering temperature. The iron melts and leaves the kiln through a tap hole while the clinker floats on the surface of the iron. The discharge of the clinker is as usual. It is subjected to a magnetic separation. The second article brings a discussion of the process by R. Durrer.

Novel high pressure grouting methods seal fissures in wet tunnel

C. R. RANKIN (Metropolitan Water District of Southern California). *Engineering News-Record*, Vol. 116, No. 19, May 7, 1936, pp. 661-663. Reviewed by N. M. NEWMARK

Inflows of water developing suddenly have added to the difficulties encountered in advancing the east Potrero heading of the San Jacinto tunnel on the Colorado River aqueduct. Different procedures have been used to seal the fissures through which the inflow occurs. Features of the grouting processes are the high pressures (up to 1500 p.s.i.), the use of sawdust for filling and checking the fissures, and the use of timber bulkheads to seal off large inflows during the grouting process. In some cases shotcrete rings have been placed in the fissured zones after heading operations have gone on beyond.

Germany builds superhighways

JOHN S. CRANDELL (Professor of Highway Engineering, University of Illinois).

Engineering sets high mark on German superhighways

I. GUTMANN (*Associate Edtior*, Engineering Index).
Engineering News-Record, Vol. 117, No. 9, Aug. 27, 1936, pp. 289-295.
Reviewed by N. M. NEWMARK

These two articles describe general features of the German government's 5000 mile system of four-and six-lane concrete express roads now under construction. The roads are said to be designed for a speed of 112 miles per hour, and the total cost has been figured at about $800,000,000 including bridges, viaducts, and grade-separation structures. The construction is thoroughly mechanized and has resulted in the development of new machines some of which are described.

Special cements for mass concrete

J. L. SAVAGE, (Chief Designing Engineer of the Bureau of Reclamation) Published by Bureau of Reclamation, 230 pages, 75 cents.

The book summarizes available information and experience in this country on the subject. It puts into concise form knowledge gained from the construction of Boulder Dam and experiments which preceded this construction and the construction of Grand Coulee Dam. These investigations have reduced to a minimum the skrinkage stresses and virtually have eliminated cracking of large concrete masses and have minimized the effect on concrete of the disintegrating elements.

The book contains 230 pages and 78 figures and formulas. It is the first time that these data have been made available.

The booklet was printed by the Bureau of Reclamation as its contribution to the Second Congress of the International Commission on Large Dams which met with the World Power Conference in Washington. The publication is offered for sale at 75 cents.

The effect of aggressive solutions upon hardened cement

A. STEOPOE, *Tonindustrie Zeitung*, Vol. 60, No. 39, 40, 487-9, 503-6, May 14, 18, 1936.
Reviewed by A. E. BEITLICH

The silica which is present is hardened cement combined as calcium hydrosilicates, is transformed slowly into acid-insoluble form by the action of aggressive solutions. The effect of sodium chloride and magnesium salt solutions of the concentrations of normal sea water, upon porous hardened cement were investigated. Sodium

chloride solution has a continuous effect of rendering silica insoluble and takes place every time the action of the solution is repeated. In the case of magnesium sulfate, the silica which is rendered insoluble first by the action of the salt, becomes soluble again due to the formation of soluble magnesium hydrosilicates. An exchange of sodium and calcium ions takes place to very slight extent only. The initial effect of aggressive solutions upon hardened cement may be considered as an ionic reaction. The presence of sodium chloride increases the aggressive activity of such salts, which have no similar ions with the sodium chloride. The presence of sodium chloride decreases the amounts of sulfur trioxide and magnesia which are retained from solution by the cement. This effect is strongest in the case of sulfur trioxide.

Cores used to control concrete for first time in Pasadena schools

Southwest Builder and Contractor, July 10, 1936. Reviewed by J. C. PEARSON

New core-cutting equipment developed by the Frank L. Howard Engineering Co., Los Angeles, adapted for taking cores at any angles and even from over head ceiling slabs, has made possible the determination of the quality of concrete in place at reasonable cost. This system of control was used on three buildings of the Pasadena Junior College, and it has demonstrated that the concrete in place is more uniform and generally of higher strength than that shown by the usual control cylinders cast from the freshly mixed concrete. In addition, visual inspection of the cores gives information on the distribution of mortar and aggregates, whether proper densification and freedom from segregation has been obtained, and indications of inadequate curing, if any. Description of the equipment is supplemented by photographs taken from the Pasadena job. Actual cost figures are not given, but experience to date has shown that the cost of control by cores instead of by cylinders, on jobs requiring 1500 yards or more of concrete, varies from a slight saving to 25 to 50 per cent more, depending on the story height of the building.

Uniform determination of the amount of shrinkage of cement for road construction

OTTO GRAF, *Zement*, Vol. 25, No. 19, May 7, 1936, pp. 317-322. Reviewed by L. T. BROWNMILLER

The essentials of procedure for measuring the shrinkage of cement for road construction have been reported in *Zement* 1935, p. 347. Naturally, the methods of measurement are constantly being revised and the following experiments have been completed since 1935 to bring about uniformity in the testing procedure:

(1) Experiments on: the preparation of the forms; (2) the quantity of mortar to be put into the form and its subsequent compacting; (3) on the handling of the prisms during hardening; (4) on the type of dry storage after the 7th day; (5) on the measurement of changes in length; (6) on the variables encountered when the tests are conducted by different engineers and assistants; also on testing in different localities; (7) on the effect of the length of storage of the cement prior to making the specimens; (8) on the relation of the shrinkage in small specimens to shrinkage in large specimens; (9) on the swelling of mortars made with various cements.

Measurements are reported in tables. And the effects of the experimental conditions are given with the idea of eliminating any variables which appreciably affect the measurements.

Questions of the systematic designation of portland cements

SERBAN SOLACOLU, *Zement*, Vol. 25, No. 24, June 11, 1936, pp. 403-407 and No. 25, June 18, 1936, pp. 419-422 Reviewed by L. T. BROWNMILLER

In the past cement has usually been considered as a constant in proportioning

concrete mixtures. This erroneous idea is gradually being displaced with the introduction of cements for special purposes. However, it now becomes important to have specific designation for the various types of cements. Author has classified portland cements with the aid of diagrams of the M_{Si}-M_{Ai} system, where M_{Si} is the S_{iO_2}: R_2O_3 modulus and M_{Ai} the Al_2O_3—Fe_2O_3 modulus. The diagram shows by curves, the mixtures of similar sintering temperatures, of similar high early strength, of similar heats of hydration and the fields of cements which are resistant to sea water. Portland cements are arranged in the following groups: (1) ordinary portland cement, with M_{Si} = 1.8—2.5 and M_{Ai} = 1.5—2.4; (2) high silica portland cements with $M_{Si} > 2.5$ and M_{Ai} = 1.5—2.4. These cements have a lower heat of hydration and higher burning temperature than (1). (3) Ferro portland cements with $M_{Ai} < 1.5$. These cements have higher strengths, lower heats of hydration, less shrinkage, lower burning temperatures and greater stability in sea water than (1). (4) Alumino portland cements with $M_{Ai} > 2.4$, which are less sensitive to excess water than (1).

Portland cement—effects of catalysis and dispersion

H. L. Kennedy, *Industrial and Engineering Chemistry*, Industrial Edition, Vol. 28, No. 8, pp. 963-969
Reviewed by Roy N. Young

This paper deals with the material TDA as a catalyst and dispersing agent for use with portland cement. The development and nature of the material is discussed and numerous data are given in the form of tabulations and graphs whereby comparisons can be made between cements treated with TDA and untreated cements. These results pertain to the latest development of the product which consists of "a mixture of triethanolamine salts and highly purified soluble calcium salts of modified lignin sulfonic acids."

The effects of the use of this material varied to a considerable degree among the cements tested, but in the large majority of cases definite advantages were indicated through increased concrete strength up to 28 days, greater durability in freezing and thawing tests, less permeability, better warehousing qualities of the cement and greater grindability of clinker during manufacturing. Furthermore it does not cause additional entrainment of air in concrete as is the case with some other grinding aids.

An accelerated determination for the sulfate resistance of portland cements

H. McC. Larmour, Eugene McMaster and Otto C. Frei, *Rock Products*, Vol. XXXIX, No. 6, June, 1936, pp. 46-48.
Reviewed by Roy N. Young

Considerable discussion and numerous data are presented, pertaining to the development of an accelerated test for the determination of the sulfate resistance of portland cement. Briefly the test consists of treating 1:3 mortar specimens (.9 w/c) as follows:

Store in damp air 24 hours; immerse in a 5 per cent solution of sodium sulfate in sealed cans to prevent any reaction with CO_2, and at 14 days subject to 400° F. atmosphere for 24 hours. Cool slowly for 6 hours and again store in the solution in a closed container for 6 days. On the twenty-first day the above cycle is repeated.

The degree of resistance offered to the attack by the sulfate solution may be indicated by observation or by comparing the crushing strength of the specimens with specimens treated similarly except stored in water. The time required for a cement to become affected through the action of the sulfate water is shortened considerably by the drying treatment at 400° F. as compared with specimens stored continuously in sulfate water at 70°.

Determining the resistance of portland cement to sulfate waters

R. W. STENZEL, *Industrial and Engineering Chemistry*, Analytical Edition, Vol. 8, No. 4, July 15, 1936, pp. 263-266. Reviewed by ROY N. YOUNG

The apparatus and method proposed for determining the resistance of portland cement to sulfate waters are described in detail. Graphs show results as affected by strength of sodium sulfate solution and water-cement ratio. Also results of tests of a number of cements of various types indicate a close correlation between the results of the sulfate resistance test and compressive strength at 6 months of concrete specimens stored under similar conditions. The compound composition was not a dependable index of the ability of the cement to resist the action of sulfate water, based on the results of the compression tests of concrete.

The proposed test is based on the warping effect of a 10 per cent sodium sulfate solution upon a neat cement slab 2 x 4.5 x 0.125 in. The degree of warping effect is measured by a spherometer.

A substitution of magnesium sulfate for sodium sulfate gave results of the same order, whereas sodium chloride showed no appreciable effect. Tests of the same sample on different days demonstrate very good reproducibility of the test.

Fifty-six bridges to span new power canal

JUSTIN NICOLET. *Engineering News-Record*, Vol. 117, No. 6, Aug. 6, 1936, pp. 181-185. Reviewed by N. M. NEWMARK

A concrete rigid frame bridge having a 109-ft. center span and counter-weighted 26½-ft. cantilever side spans is among the 56 bridges over the new 33-mile power canal of the Loup River Public Power District in Nebraska. This long span rigid frame bridge carries a roadway 24 ft. between curbs on six heavily reinforced ribs 15 in. wide, 42 in. deep at midspan, and 84 in. deep at the ends. A hinge detail is provided at the base of the legs or columns of the structure, concentrating the bearing on lead plates over the center 18 in. of the column, the outer parts of the column resting on cork fillers. Inclined hinge bars pass through the lead plates and connect the column with the footing. Concrete counterweights weighing about 105 tons each are suspended between and poured monolithically with the cantilever girders.

Four 2-span concrete rigid frames with span lengths 57 to 60 ft. long carrying a 22 ft. roadway are described also. Hinges similar to those used in the long span bridge are provided at the bases of all the columns. The analyses included determination of stresses due to temperature variation of ± 60° F., shrinkage and plastic flow equivalent to a drop in temperature of 45° F., and a settlement of any column of ¼ in. These effects constituted 24 to 34 per cent of the total stresses.

The working stresses permitted are 24,000 p.s.i. for steel and 1275 p.s.i. for concrete. The concrete used has a 28-day cylinder strength of 3500 to 4500 p.s.i.

Effect of temperature on asbestos-cement manufacture

R. ITSCHNER. *Cement and Cement Manufacture*, Vol. 9, No. 7, July 1936, pp. 157-8, (Abstracted from a recent number of *Zement*.) Reviewed by J. C. PEARSON

Experience has shown that the output of cement-asbestos shingles per ton of cement is higher in summer than in winter. This is apparently to be explained by the fact that more $Ca(OH)_2$ is formed by the higher rate of hydrolysis at elevated temperatures, and this requires more water and gives an increased yield. Generally speaking, there is very little accurate information available concerning the $Ca(OH)_2$ content of set asbestos cement. Author shows how this may be determined by gas or gravimetric analysis, and states that several hundred samples thus examined showed variations of about 30 per cent. The $Ca(OH)_2$ acts only as a filler and causes

efflorescence when dissolved by rain or dew, thus spoiling the appearance of colored surfaces. One means of overcoming efflorescence, widely used in the industry, is treatment with CO_2. Data are given showing the rate of gas absorption by the shingles, which is very rapid at first but is very slow after 6 or 8 hours. This treatment is carried out in a closed chamber, in which the rise of temperature produced by the reaction is helpful in removing some of the water formed, which allows the gas to penetrate further. It appears that about 500 g. of CO_2 per sq. meter of cement asbestos sheet (calculated for 1 side) is sufficient to neutralize the $Ca(OH)_2$ to a depth of some tenths of a millimeter, and thus prevent efflorescence. Author states that this increases the strength by about 20 per cent, but it cannot be taken advantage of in shortening the curing because the hardening process of the cement must be carried to such a point that temperature and humidity differences on the upper and lower surfaces of the shingles will not cause distortion.

Volume changes in concrete under various storage conditions

H. Kayser and G. Buth, *Beton und Eisen*, Vol. 35, No. 10, pp. 169-175, May 20, 1936.
Reviewed by Inge Lyse

An investigation was made to ascertain the volume change behavior of concrete subjected to different types of waters which have been found to cause disintegration of concrete. The test specimens were 2¾ x 2¾ x 27½ in. bars with bronze lugs for observations. The mix was 1:3:3 by volumes with sufficient water for plastic consistency. The aggregate consisted of sand and gravel. Two cements were used, one of them with and one without admixture. In addition to air and water curing, submersion in a 5.00 per cent sodium sulphate solution and a 0.25 per cent gypsum solution were used. The results showed that:

1. The shrinkage in air was essentially equal for the three cements used and reached an optimum value after about ten weeks. The subsequent volume variations were principally due to temperature and humidity changes.
2. The relative humidity was found to have a marked effect upon the shrinkage.
3. Portland cement concrete with and without Sika admixture showed a slight swelling upon submersion in water and soon reached a stationary volume. High alumina cement concrete, however, showed continuous expansion during water storage.
4. The specimens stored in the sulphate solutions showed continuous expansion with smaller values for the weak gypsum solution than for the stronger sodium solution. The specimens having portland cements disintegrated to such an extent that no observation could be taken after four years in the gypsum solution and after one and one-half years in the sodium solution. The high alumina cement showed greater resistance in these solutions than did the portland cements.

Notes on Method of Waterproofing Large Dams

M. Grelot (Engineer in Chief of Ponts et Chaussees) and M. Chalos (Engineer of Ponts et Chaussees)
Les Annales des Ponts et Chaussees, 1935, VII, July, pp. 177-186. Reviewed by B. Moreell

The authors state that methods of consolidating and waterproofing the foundation materials for large dams by modern methods of injecting foreign material have given entirely satisfactory results. However, the waterproofing of the masonry of existing dams has been a more difficult procedure and methods proposed have in many instances proven ineffective. They describe a method which they designed for waterproofing an existing stone masonry dam, the mortar joints of which had deteriorated to an extent such as to affect the stability of the dam.

The method consists of placing over the entire upstream face of the dam a blanket of reinforced concrete, varying in thickness from 1 meter at the top to 8.9 m. at the base. At approximately 70 cm. from the upstream face and parallel thereto, there is embedded in this blanket of concrete a sheet metal diaphragm fabricated in place by means of gas welding. The sheet metal diaphragm is anchored to the masonry of the upstream face by means of flat iron bars which are embedded in the masonry but which are sufficiently flexible to permit movement in a vertical direction.

Ample provision is made for expansion and contraction of the sheet metal diaphragm by the arrangement of the anchor mentioned hereinbefore, and also by vertical expansion joints spaced 10 meters on centers. The concrete blanket is made discontinuous at the vertical expansion joints, the spaces between the sections being filled with asphalt and made watertight by a crimped copper sheet.

The authors do not give the location of the dam and state that the work has not yet been undertaken. They indicate that welding of the sheet metal diaphragm permits a successful application of this method.

Mortar and concrete with portland-puzzolan cement

A. POULSEN, *Ingenioren*, No. 29, May 13, 1936. Reviewed by INGE LYSE

This article presents a critical discussion of the paper: "Properties of Mortars and Concretes Containing Portland-Puzzolan Cements" by Davis, Kelly, Troxell and Davis, JOURNAL of the Amer. Concrete Inst., Sept.-Oct. 1935. The author takes issue on several important points. He is surprised to note that the investigation could merely establish the optimum puzzolan content within the range of from 10 to 30 per cent. He questions this "oracular answer" which in his opinion should at best be within the limits of 20 to 50 per cent. Moler (diatomateous earth) cement contains 25 per cent admixture, and trass has been used in quantities greater than 40 per cent. Criticism is also made of the use of 20 per cent by weight of all the various types of puzzolans in the investigation. It is pointed out that not only the variation in silica content but also the nature of the silica in the puzzolan is of importance. A certain percentage silica by weight would have been a more logical basis for comparison, but even this would only be partly correct because only the amorphous portion of the silica is active. Every puzzolan contains in addition to the soluble silica, a quantity of waste material, which if large, will weaken the mortar. However, a puzzolan which is rich in amorphous silica, such as diatomateous earth, will, when added to cement, produce a stronger cement than the original portland cement. A puzzolan's value may readily be analyzed simply by ascertaining how much of it is soluble in alkalies. The author has for 25 years pointed out that this chemical condition determines the value of a puzzolan and he is somewhat disappointed to see that others look at the matter in a different light. He feels certain that if the investigators had based their studies on the amount of soluble silica present in the various puzzolans their conclusions would have been different. The author finally maintains that the question of economy in the manufacture of the various cements can only be determined in the mills.

A Study of the quality, the design and the economy of concrete

INGE LYSE, Reprint from *Journal of the Franklin Institute*, May, June, July, 1936.

The following is from the author's introduction:

The knowledge of concrete making has made a remarkably rapid advance during the last 15 years—principally brought about by stimulus given to scientific investi-

gations by the publication of the water-cement ratio law by D. A. Abrams. Investigations into the properties of concrete and of its ingredients brought forth a great variety of results, the interpretation of which led to many different theories: the cement-void ratio, the cement-space ratio, the water-cement ratio laws, and many others. Correlation of the different theories, harmony of the different results was lacking. "The main purpose of the study presented in this paper is to correlate some of the most important developments and to attempt to bring the whole problem of concrete and its various qualities in under one single conception. . . . the cement concentration law for strength and other qualities of concrete." There was no generally accepted basis for designing concrete mixes. In this paper a method of design recently developed at Lehigh University will be presented because of its simplicity and other advantages—setting forth a relationship between quality and economy of concrete. "Although this relationship came in as an unforeseen consequence of the other problems, it probably is the most outstanding of all in its practical aspects because it deals with the most fundamental of all practical problems, namely that of designing on basis of economy." The paper is divided into three major parts, one on Qualities, one on Design, and one on Economy of concrete.

The author's summary:

"1. The strength and practically all other desirable qualities of concrete except volume changes, increase uniformly with the increase in cement concentration in the amount of water in the concrete at the time of hardening.

"2. For a given type and gradation of aggregates the net amount of mixing water in the concrete remains practically constant as long as the placeability of the concrete remains approximately constant. Changes in richness of mix may therefore be accomplished by simply substituting g_a/g_c pound of aggregate for each one pound change in the cement content.

"3. The economy of plain concrete increases very markedly with the increase in strength, with variables such as gradation of aggregates and strength-quality of cement less important.

"4. The cost ratio between steel and concrete for equal economy in reinforced concrete columns is: $p_c/p_c' = 0.18\ (f_s/f_c')$ forwelded reinforcement and $p_s/p_c' = 0.15\ (_s/f_c')$ for spliced reinforcement.

"5. The strength of the concrete has relatively small effect upon the economy of reinforced concrete members subjected to flexure.

"6. The balanced reinforcement approaches the most economical design for present-day cost ratios and ordinary strength of the concrete.

"7. The economy of the reinforcing steel is directly proportional to the ratio between its unit price and yieldpoint stress."

Application of the curve of intrinsic resistance of concrete to the rational design of reinforced concrete

M. Chalos (Engineer of Ponts et Chaussees), *Les Annales des Ponts et Chaussees*, 1935, VII, July, pp. 76-95. Reviewed by B. Moreell

The author describes, in detail, the theory of the "Curve of Intrinsic Resistance," the use of which for the design of concrete and reinforced concrete was first proposed by the "Syndicate of Reinforced Concrete Constructors of France" in 1930.

The curve is based on the assumption that concrete under combined stress behaves like a homogeneous and isotropic material. The "curve of intrinsic resistance" is the envelope of the Mohr's circles drawn for the principal stresses at failure. It is stated that concrete in the body of a structure may be subjected to elastic stress in

any direction. When the stress on any element reaches the limiting value determined by the curve of intrinsic resistance, there will be failure.

The author develops a general formula for the curve of intrinsic resistance for concrete based on experimental results, and then proceeds to establish formulas for the determination of the constants of the general formula, which he states is a semi-cubical parabola.

After having determined the formulas for the constants, he presents tables giving the constants for the "curve" for any particular concrete. These tables are based on the values of the ultimate tensile and compressive strengths of the concrete. After the tables, the author gives typical examples to show how the determination of maximum stress is made in the actual design of members subjected to bending and direct stress. He demonstrates that the fibre subjected to the highest stress in the case of a member subjected to direct stress, bending moment, and sheer is not the extreme fibre but is a fibre located at approximately mid-height of the section for the conditions of his problem. He demonstrates that this method of analysis enables the designer to make a more rational distribution of the shear reinforcement and thus to prevent excessive cracking.

He analyzes the case of a member subjected to bending and compression and the same member subjected to bending and tension, and concludes that for the latter case the value of dangerous shearing stress is much smaller than for the former. This has practical application since, as a result of shrinkage, reinforced concrete is almost invariably in a condition similar to that of a member subjected to bending and tension. The author concludes that this condition calls for the provision of secondary (shear) reinforcement for members subjected to bending, even though the usual methods of design do not indicate the necessity for such reinforcement. He concludes that, in any event, it is desirable to determine the stability of reinforced concrete members subjected to bending and shearing stress by investigating the direction and intensity of the maximum elastic stress.

Consideration of the effect of combined stress is a refinement not usually encountered in American practice. The propriety of the assumption that concrete under combined stress behaves like a homogeneous and isotropic material has been questioned by other investigators. An interesting discussion of this feature can be found in "A study of the failure of concrete under combined compressive stresses," by Richart, Brandtzaeg and Brown, bulletin 185, Engineering Experiment Station, University of Illinois.

This reviewer is of the opinion that serious consideration of the phenomena of combined stresses would be justified, particularly in massive concrete structures.

Proposed specification for aggregates and water for the making of concrete

Annales de L'Institut Technique du Batimet et des Travaux Publics; Vol. 1, No. 2, 1936.

Reviewed by P. H. BATES

This specification is being considered by a commission in France, and in its present form is open for discussion. In our terminology, it might be called a proposed tentative specification.

In addition to covering the requirements for aggregates and water for making concrete, it includes the test methods to be followed in testing materials to determine whether they will meet the requirements. The first article defines aggregates and presents three classes:

Class A—For use in making concrete to develop high strengths.
Class B—For use in making impermeable concrete.
Class C—For lower strength or more permeable concrete than demanded in A and B.

Article 2 limits the impurities which are separated into a number of groups: pro-

hibited impurities such as coals, ashes, fine clay, etc.; impurities permitted within certain limits such as sulfates and sulfides oxidizable to sulfates which are limited to a 0.25 per cent equivalent of sulfuric anhydride for Classes A and B and one per cent for Class C; organic impurities which are similar to those covered by the United States tests for soluble organic matter, and finally, finely divided material such as clay, silt, etc., which are limited to 3 per cent in Classes A and B and 5 per cent in Class C.

The general characteristics of the aggregate are covered by such terms as "sharp," "angular," etc., and in addition there are a number of rather interesting and unusual requirements. The porosity of Classes A and B shall not exceed 5 per cent and 10 per cent for Class C. The sieve sizes indicated extend from 2 to 100 mm., each size being twice that of the previous one. The strength requirements are specified both in tension and compression. The strengths of any aggregate of Classes A and B shall be higher than the strengths with a standard aggregate when the same cement is used; for Class C, the strength should be at least 0.8 of that obtained with standard aggregate. There is also a requirement that the mean volumetric coefficient of the particles retained on a sieve having a modulus of 38 should be higher than .15 for Classes A and B and .12 for Class C. The modulus just stated is—

$$M = 10 \times (3 + \log d)$$

The modulus of the aggregate as a whole is determined from the equation

$$M = a_1 m_1 + a_2 m_2 + \text{— — — —} a_n m_n$$

where a_1 etc. is the percentage of each size

m_1 etc. is the modulus of each size calculated from the formula

$$M = \frac{\text{modulus of } d + \text{modulus of } 2d}{2}$$

the modulus of d being $= 10 \times (3 + \log d)$, d being the diameter of the size expressed in microns.

The test methods, etc. are given in considerable detail. The original article should be referred to by those who are particularly interested in the specifications for aggregate. It is interesting to note the extreme detail being followed in France in the requirements for aggregates and water especially in view of the fact that in the United States the tendency seems to be to buy such materials under relatively loose specifications.

Deterioration of concrete owing to chemical attack

F. M. Lea. *Cement and Cement Manufacture.* Vol. 9, No. 6, June, 1936, pp. 130-138.
Reviewed by J. C. Pearson

The resistance of various types of cement to chemical attack differs widely owing to the different reaction products formed as they react with water to set and harden. In portland cement it is generally free $Ca(OH)_2$ and hydrated calcium aluminate which react with the aggressive agent. In portland blast-furnace cement the compounds are similar, but the lime is lower, and there is less free $Ca(OH)_2$. In puzzolanic cements an opportunity is given for the free $Ca(OH)_2$ to form more insoluble compounds, and after maturing, the free $Ca(OH)_2$ may be almost zero. In aluminous cements the hydration products are principally compounds of lime and alumina with free hydrated alumina. This latter compound is relatively inert to chemical attack, and probably forms a protective film over the more susceptible compounds. It seems that corresponding films may be formed in puzzolanic cements which also add to the resistance of this type.

Soluble sulphate salts are common in certain soils, natural waters trade wastes and sea water. They react with free $Ca(OH)_2$ and the hydrated aluminates, forming compounds that cause expansion and distintegration of concrete. The resistance of portland cement concrete to sulphate attach increases markedly with the density

and impermeability of the material, and aged concrete is more resistant than new concrete. Aluminous cement concrete, properly made, seems to be practically immune to sulphate attack. Puzzolanic cements are intermediate in resistance, but are more resistant than portland, and should have a considerable field of usefulness under the less severe sulphate exposures. Portland blast furnace cement, either alone or blended with puzzolan has been widely used in Europe for sea water work. Author suggests upper limits of .15 per cent SO_3 as gypsum in clay soils, and .015 per cent SO_3 as gypsum in ground waters as unlikely to cause damage to well made portland cement concrete.

Pure waters have a solvent action on set cement and concrete tending to dissolve the lime and leave a non-coherent residue. Ordinarily this action is slight and does not penetrate into the mass. It may, however, be serious with lean concretes and wherever seepage may occur. Waters made acidic with dissolved CO_2, organic acids, and sulfuric acid from polluted atmospheres or from pyrites in soils, are more aggressive than pure waters. Aluminous cement concrete has been found by experience to be nearly immune to the attack of low-acid waters, and the author attributes to puzzolanic cements a higher resistance to such waters than portland cement.

Under certain conditions sewer gas may contain appreciable quantities of H_2S, which is a potential source of injury to concrete and mortar. In the presence of oxygen this gas forms sulphurous and sulfuric acids which convert the lime compounds to calcium sulphate. This attack may be very severe in long outfall sewers, and occurs above the level of the liquid. It has been difficult to find suitable construction materials for such structures, as protective coatings are of short life, and even aluminous and sand-sulphur cements are not immune.

Industrial wastes may be injurious from content of organic matter or mineral matter. Saponifiable fats and oils can cause disintegration of concrete, but injury is more likely to occur from organic wastes which are acidic. No cement product can be expected to withstand continued exposure even to very dilute solutions of mineral acids and any waste containing these is a potential source of danger. Salts in wastes may vary greatly, and in general chlorides, carbonates, nitrates and phosphates are not likely to be injurious in low concentrations. Amonium nitrate, however, is aggressive in concentrations of .5 to 1 per cent or more. The sulphates are bad and their effect increases with concentration up to 1 per cent, but more slowly thereafter. Ammonium sulphate is the most aggressive of all the sulphate salts in its effect on concrete.

Design for grouting at Boulder Dam

A. V. WERNER, *Civil Engineering*, Vol. 6, No. 9, Sept., 1936, p. 572 Reviewed by J. R. SHANK

417,000 cu. ft. of cement were used and 304000 lin. ft. of holes were drilled for the grouting of the foundations and structures of Boulder Dam. Cement screened through a 200 mesh sieve and very little or no sand were used in the mixtures. The water-cement ratios by volume ranged from 1 to 10. Sand and unscreened cement was used only for filling large voids at low pressures. Air operated, duplex, piston displacement "slush" pumps were used. The pressure ranged from 50 to 3000 p.s.i.

The grouting may be divided into three divisions, low pressure grouting, high pressure grouting, and contraction joint grouting.

Low pressure grouting at from 50 to 300 lb., in holes from 30 to 50 ft. deep, was used to prepare rock areas for grouting at high pressures, for voids above concrete linings in tunnels, and adits, and in the concrete fillings of plugs and anchors, and around penstock pipes. The thicker mixes were used. The preparatory low pressure

grouting was done on the rock under the up-stream face before any concrete was placed. The holes were spaced about 20 ft. both ways. The same was done under the four intake towers.

High pressure grouting, at from 500 to 1000 lb. pressure (maximum 3000), and in holes as much as 150 ft. deep was used for the main grout curtain along the line of the axis of the dam. The holes were spaced about 5 ft. apart and were drilled from the drainage gallery at 15 degrees from the vertical, in the up-stream direction and on radial planes. It was required that none of these holes be grouted before 100 feet of concrete had been placed above the hole, the concrete cooled, and the contraction joints grouted. High pressure grouting was also applied to holes under the intake towers drilled from the surface and from the diversion tunnels. These holes were at considerable angles with the vertical. Another sustem of high pressure grouting was carried on at the upstream heel of the dam through holes drilled at about 30 degrees with the vertical also on the radial planes but sloping downstream. These holes were drilled to within 20 feet of the line of high pressure grout curtain holes at the axis of the dam. Where concrete had to be placed before grouting could begin, pipes were left in the concrete through which to work.

Grouting contraction joints was done similar to that at the Owyhee and Gibson dams. The block sizes in the dam proper varied from 25 x 30 to 50 x 50 ft. Metal grout stops were placed across radial joints at both faces of the dam and across the circumferential joints at each junction with a radial joint, and every 50 ft. vertically. The grout was forced at 50 lb. pressure into lower headers until the same consistency of grout was forced out at upper headers. All joints were filled with water before grouting was started to guard against displacement of the blocks and to prevent infection of an ungrouted joint by grout leakage. The abutment contact planes were provided with a similar grouting system to provide for later grouting.

The experience of this work together with the large amount of preliminary investigation on other dams brought out a number of valuable points. Extensive exploratory drilling and hydraulic tests on drill holes are valuable. The depth of the hole should be from 25 to 30 per cent of the hydrostatic reservoir head above the foundation level of the hole. The axis of the hole should be normal to the foundation surface. Holes should be drilled with a rotary type drill. More effective distribution is secured by first drilling and grouting alternate holes of a one-line arrangement; or the down-stream line of a two-line arrangement. Preferably, holes should not be drilled or left open closer than a distance of four hole spaces from a hole being grouted. In working with deep holes the best results are secured by alternately drilling and grouting in depths of from 15 to 20 feet. A thin grout with a water-cement ratio of from 7 to 5 should be used for the initial injection for deep hole, high pressure grouting. The mix should be thickened and the rate of pumping increased in accordance with the capacity and behavior of the hole. The maximum amount of grout should be forced into the hole in the shortest possible time. Supplemental grouting may be required after the dam is tested under full load.

Life expectancy of concrete pavements

Harold Allen (Engineer of Materials, Kansas State Highway Commission) 1936. Highway Research Board. Highway Research Board Abstracts

Life expectancy was estimated by comparing the cycles of freezing and thawing and range in temperature of concrete pavements during one season with laboratory freezing and thawing tests on cores from pavements.

To determine the number of cycles of freezing and thawing and the temperature range encountered by concrete pavements during one winter, recording thermo-

meters were installed in the top, bottom and middle of the concrete pavement slab on an experimental paving project east of Lawrence on Kansas Highway No. 10. Thermometers were installed to measure the subgrade and air temperatures. Table 1 was compiled from the temperature records of Nov. 1, 1935 to March 1, 1936. A cycle of freezing and thawing was considered to be any change below 32° F. and back to that temperature. For example, if the thermometer dropped to 31° and then came back to 32° the change was counted as a cycle. If it dropped to 10° below zero and returned to 32° above, that period was also counted as a cycle. The range in temperature recorded is the number of degrees change below 32° F. With these data it was possible to compare the effect of natural freezing and thawing with that of the artificial freezing and thawing used in the laboratory.

In the Kansas State Highway Laboratory, specimens lowered in temperature from 70° F. to —15° F. which is a change of 85°. In the field, the average change was 8°. If it be assumed that the volume change of concrete per degree of temperature is constant, the volume change due to laboratory change of temperature of 85° is ten times that caused by a natural change of 8°. On this basis, therefore, 75 cycles of laboratory freezing and thawing are comparable to 750 cycles of natural freezing and thawing. Since there were 35 cycles in the top of the slab and 20 cycles in the bottom of the slab, we may assume for easy calculation an average of 30 cycles per year. Therefore, the life expectancy of pavement concrete which would withstand 75 cycles of laboratory freezing and thawing would be 750 divided by 30 or 25 years.

TABLE 1—CYCLES OF FREEZING AND THAWING AND RANGE IN TEMPERATURES NOVEMBER 1, 1935 TO MARCH 1, 1936

Location of Thermometers	Cycles	Range of Temperature below 32° F.	
		Maximum	Average
Air	49	43	
1 Inch from top of slab	35	30	8
1 Inch from bottom of slab	20	25	8
Subgrade	8	8	

In Table 2 is recorded the number of cycles necessary to cause failure in concrete cores taken at random from Kansas pavements. Using the foregoing premise, the number of laboratory cycles of freezing and thawing multiplied by 10 and divided by 30 gives the life expectancy of the concrete. These data cover only freezing and thawing; other factors that may affect the life of concrete pavement were not taken into account. It is also pointed out that the data are for only one winter.

TABLE 2—FREEZING AND THAWING DATA ON CONCRETE CORES

Core Number	Cycles at Failure	Life Expectancy in Years
1	70	23
3	140	47
5	130	43
7	125	42
9	50	16
11	40	13
13	100	33
14	100	33
15	100	33
16	70	23
18	70	23
19	70	23
20	100	33
22	132	44

Use of mixtures of alumina and slag cements as fast setting hydraulic binders

R. VUILLEMIN, *Le Genie Civil*, Vol. CVIII, No. 18, pp 416-418. Reviewed by R. L. BERTIN

When alumina cement was first introduced, it was found that combining it with portland or slag cements and even lime, accelerated the set of the latter and produced a mixture which acted very much like those made with prompt cement. Such practice was not without danger since it is well known that the addition of even small quantities of other binders to alumina cement can accelerate its set to a point where it will harden during mixing. This paper gives the results of tests to learn the resistance of mortars containing mixtures of alumina or slag cements in various proportions, with a view of promoting their use.

Two kinds of slag cements are manufactured in France, one having a lime base which contains at least two thirds of granulated slag and at most one third of lime ground together cold. The other is a mixture of at least 85 per cent of granulated slag and at most 15 per cent of portland cement being frequently obtained by passing through a rotary kiln a mixture of calcareous rock and slag, the latter replacing the clay. The former type of slag cement was used exclusively.

The advantages of the mixtures of cements and alumina cements over the prompt and semiprompt cements are stated as follows:

(1) Within certain limits, the resistance of mortars and concretes containing these mixtures is of the same order as that of the cements to which the alumina cement is added.

(2) The mixtures do not deteriorate as much as the prompt cements in storage.

(3) By varying the amount of alumina cement used in the mixture, a wide range of setting time is obtained meeting all practical requirements. (See Table 1).

The preference given the lime base slag cement over portland cement is justified as follows:

(1) The cost is less than that of portland cement.

(2) For a given setting speed, a much smaller proportion of alumina cement is required.

(3) Slag cement offers greater resistance to the action of pure water.

One set of tests covers the tensile resistance of 1-3 mortars at 1, 2 and 7 days, the proportions of alumina cement to slag cement varying from 1 to 1, to 1 to 10.

While the resistance of the mixtures are lower than for pure alumina cement, they are greater than that of a prompt cement. It is noted also that the resistance does not increase with the quantity of alumina cement in the mixture.

A second set of tests covers the influence of the length of mixing period on the strength for varying proportions of the two concretes. From these tests the following rule was derived:

"The maximum time for use after adding water to a mixture of alumina and slag cements is approximately one half the period of initial set as measured by the Vicat needle."

The rule finds application in many cases, notably where cement plastering or waterproofing is done at high altitudes, taking advantage of the hours during the day when the temperature is above freezing, or executing work between tides. For repairing leaking structures, there is advantage in using a cement gun in which case a concentration of alumina cement may be used such as to give an initial set of the order of one minute.

TABLE 1—INITIAL AND FINAL SET OF 1-3 MORTARS FOR DIFFERENT MIXTURES OF ALUMINA AND SLAG CEMENTS

Mixture		Setting Time in Minutes	
Alumina Cement	Slag Cement	Initial Set	Final Set
1	1	1.25	2.25
1	2	2.00	3.75
1	3	3.00	6.00
1	4	4.00	9.00
1	5	5.00	11.50
1	6	6.00	13.25
1	7	6.75	14.50
1	8	7.50	.....
1	9	10.50	.....
1	10	15.00	.....

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . NOVEMBER-DECEMBER 1936

Resistance of Cement to the Corrosive Action of Sodium Sulphate Solutions*

By Lewis H. Tuthill†
MEMBER AMERICAN CONCRETE INSTITUTE

PURPOSE OF TEST PROGRAM

In certain portions of the location of the Colorado River aqueduct and distribution system, soils and soil waters contain sodium sulphate, and other salts less corrosive to concrete, in amounts which present a hazard to the permanence of ordinary concrete construction. These hazards were considered particularly important in view of the many concrete structures which have disintegrated elsewhere in the presence of such ground waters. Exceptions to such failure have been noted and have encouraged investigators to search for the elusive properties of cement or concrete composition responsible for such resistance. Consequently study of this problem was included at an early date in the activities of the testing laboratory of the Metropolitan Water District of Southern California. The problem quickly resolved itself into the questions: (1) Was there reason to believe that concrete containing any *economically available* commercial cement could be used safely in a structure that must last for generations? (2) Was there any effective additional protection that could be used efficiently to aid the cement in resisting this disintegrating force? (3) What changes in cement composition could be made which would give greatest assurance of permanent resistance to sodium sulphate solutions in typical ground water concentrations? The effort to answer the last question soon became the most significant part of the investigation, and in recent years became the entire purpose

*Received by the Institute Secretary Aug. 5, 1936. For presentation at 33rd Annual Convention, February 1937.
†Testing Engineer, Metropolitan Water District of Southern California, Banning.

of the test program, as the answers to questions 1 and 2 were recognized as negative at a relatively early date.

The results of tests supporting this early answer to question 1 are not reported because they are frequently corroborated throughout this paper.

A number of protective materials were submitted for tests and were applied to half briquets according to the manufacturers' directions. The opposite half of the briquet was similarly exposed in the unprotected condition. Although many of these applications have well withstood the attack of the 1 per cent sulphate solution for about three years, the very nature of the application procedure and equipment confines its use entirely to precast sections or interiors because there is no satisfactory way of treating the surface of concrete cast in place on the subgrade. The susceptibility of such coatings to damage from handling in the course of construction and possibility from soil stress later, also leave doubt as to the suitability or reliability of such protection. Consequently, the answer to question 2 was considered negative, and search for a cement of an inherently resistant composition remained the principal objective of the investigation.

EARLY TESTS OF SPECIAL CEMENTS

Early in 1933, when results of many tests made to obtain the answer to question 1 were available, it was evident that local ordinary commercial portland cement was incapable of resisting the corrosive action of a 1 per cent sodium sulphate solution. From this conclusion, it was apparent that, if concrete was to be used successfully where corrosive ground water was prevalent, a cement which was particularly and definitely resistant to such corrosion would have to be developed. As a consequence a series of tests, (the SS—Sodium Sulphate—series) was inaugurated in which samples of special cements which it was thought might be more resistant than ordinary cements, were tested in 20-30 Ottawa and San Gabriel sand mortars submerged in 1 per cent sodium sulphate solutions renewed semi-monthly.

The list of such special cements investigated was headed by a mill-run commercial high silica cement of which a series of samples was obtained over a period of 15 months. These samples were so taken because a group of briquets made with a small sample of this cement (then newly marketed) early in 1932 was in perfect condition after a year of sulphate exposure, while companion specimens of the four ordinary cements had completely disintegrated at 8 months. (Incidentally, these early specimens are still intact after 54 months of

exposure, although many subsequent specimens of the same high silica cement have disintegrated long ago. See Table 1).

Several samples of different lots of mill-run cement of a special composition made for the Morris and Boulder dams, and designed to produce a low heat of hydration, were obtained. The different local mills contributed other samples of cement having compositions or blends which each considered would improve or create resistance to sulphate attack. Several early strength and other cements, together with one sample each of four brands of ordinary commercial portland cement, were included for comparison.

Ottawa mortars (20-30) were made of each cement, mixed 1 to 3 by weight with a w/c by volume ratio of .90. This mix produced a plastic mortar of about 3-in. flow (using the 3-in. Burmister flow trough) varying somewhat with the brand or composition of the cement. Only six briquets were made from each of these small mortar batches. The specimens were cured in water at 70° F. until time for tension test at the age of 7 and 28 days. Following each of these tension tests, three half briquets were placed immediately in the 1 per cent sodium sulphate solution. Following the 7-day test, three half-briquets were stored in relatively dry laboratory air for 21 days before being placed in the solution. Following the 28-day test three half briquets were similarly stored in dry air for 28 days before being placed in the solution.

For the balance of this series of tests standard regraded San Gabriel sand mortars were used mixed 1 to 3 by weight with a water-cement ratio of .95. In batches using pozzolanic cements which had a specific gravity lower than the average for the ordinary cements, the weight of cement used was reduced according to the ratio of the specific gravities. The volume of water and the weight of sand remained constant for all batches. With this mix the mortars of ordinary cements had a consistency of 4-in. and the portland-pozzolan cements a 3-in. consistency, both measured in the 3-in. Burmister flow trough.

Briquets—made with the regraded sand were cured and tested as before; some halves were dried and immersed in 1 per cent sodium sulphate solutions—all similarly to the 20-30 Ottawa mortar briquets described above.

There were eight groups of three half-briquets for each cement investigated. Each group of three half-briquets was placed in an 8-oz. wide-mouth glass jar. The jars were then filled with 1 per cent sodium sulphate solution completely covering the specimens, and covered with a metal cap. The solution was replaced with a new solution of the same strength at the end of each two week period. The

TABLE 1—RESISTANCE TO 1 PER CENT SODIUM SULPHATE SOLUTION OF MORTARS USING VARIOUS TYPES AND BRANDS OF CEMENT

Note: Data tabulated are number of months exposure before reaching condition noted as follows:
O K = No evidence of disintegration at given age.
I D = Initial disintegration clearly visible at given age.
C D = Complete disintegration.

Batch No.—SS Series	SAN GABRIEL SAND MORTARS				20-30 OTTAWA SAND MORTARS			
	7 Day Cure		28 Day Cure		7 Day Cure		28 Day Cure	
	In at Once Wet	Dried 21 Da. First	In at Once Wet	Dried 28 Da. First	In at Once Wet	Dried 21 Da. First	In at Once Wet	Dried 28 Da. First
PORTLAND-POZZOLAN CEMENTS								
Z brand High Silica								
S282S1	none	none	51–OK	none	none	none	none	none
8S2	39–CD	40–ID	39–CD	37–CD	40–ID	40–ID	40–ID	39–ID
9S3	39–CD	39–CD	38–CD	37–CD	23–CD	38–CD	17–CD	24–CD
10S4	29–CD	36–CD	24–CD	30–CD	29–CD	38–CD	28–CD	30–CD
13S5	38–CD	37–CD	30–ID	none	28–CD	25–CD	23–CD	none
14S6	37–ID	none	29–ID	10–CD	35–CD	none	27–CD	10–CD
17S7	29–ID	37–OK	37–OK	25–CD	29–ID	37–OK	37–ID	24–CD
18S8	36–CD	37–OK	36–CD	21–CD	36–CD	37–ID	37–ID	18–CD
19S9	22–CD	25–CD	14–CD	14–CD	19–CD	25–CD	14–CD	13–CD
20S10	26–CD	25–CD	25–CD	13–CD	15–CD	31–CD	14–CD	13–CD
28S11	34–CD	25–CD	35–OK	29–CD	24–CD	24–CD	24–CD	16–CD
30S12	25–CD	35–ID	35–ID	22–CD	20–CD	32–CD	19–CD	15–CD
31S13	35–CD	35–CD	35–ID	31–CD	17–CD	33–CD	34–CD	15–CD
32S14	36–OK	35–ID	35–OK	34–CD	36–ID	35–ID	35–ID	25–ID
33S15	24–CD	23–CD	23–CD	15–CD	13–CD	17–CD	14–CD	33–CD
34S16	23–CD	31–CD	26–ID	15–CD	17–CD	23–CD	17–CD	12–CD
43S17	33–CD	26–ID	34–OK	15–CD	19–CD	31–CD	19–CD	12–CD
44S18	26–ID	34–ID	34–ID	26–CD	16–CD	32–CD	19–CD	12–CD
H brand Tufa (standard clinker and Tufa 70-30 blend)								
24T	25–CD	35–OK	24–CD	16–CD	17–CD	35–OK	14–CD	12–CD
25T	35–CD	35–OK	35–ID	34–CD	21–CD	35–CD	20–CD	16–CD
61T	19–CD	30–ID	30–ID	26–CD	14–CD	28–CD	9–CD	11–CD
M brand cement (special blend with standard N brand clinker)								
76A2	28–OK	27–OK	27–OK	25–CD	28–OK	27–OK	27–OK	26–OK
77B2	27–CD	27–OK	27–OK	26–CD	28–OK	27–OK	27–OK	26–ID
MORRIS DAM LOW HEAT CEMENTS								
R brand								
6VPC	41–OK	40–OK	40–OK	39–OK	40–CD	40–OK	40–OK	39–OK
11VPC	41–OK	40–OK	40–OK	39–CD	41–OK	40–OK	40–OK	39–OK
12VPC	39–OK	38–OK	38–OK	none	39–OK	39–OK	38–OK	none
15VPC	39–OK	none	36–CD	37–OK	37–OK	none	38–OK	37–OK
26VPC	36–ID	35–OK	35–OK	34–ID	36–OK	35–OK	35–OK	34–OK
S brand								
46RPC	35–ID	34–ID	34–OK	33–ID	35–CD	34–ID	33–CD	32–CD
47RPC	34–CD	33–CD	34–OK	33–ID	35–OK	34–OK	33–CD	33–OK
N brand								
1CPC	41–OK	40–OK	40–OK	39–OK		No Ottawa made		
27CPC	36–OK	35–OK	34–CD	34–OK	35–CD	32–CD	33–CD	34–OK
39CPC	35–ID	34–OK	34–OK	33–OK	35–OK	34–OK	33–CD	33–OK
40CPC	35–OK	34–OK	34–OK	33–OK	33–CD	34–OK	34–OK	33?OK
41CPC	35–OK	34–OK	26–OK	33–ID	34–CD	34–OK	34–OK	33–OK
42CPC	35–OK	34–OK	34–OK	33–OK	35–OK	34–OK	34–OK	33–OK
H brand								
21MPC	37–OK	36–OK	36–OK	35–ID	37–ID	36–ID	36–ID	35–OK
BOULDER DAM LOW HEAT CEMENTS								
R brand								
62VBD	31–OK	30–OK	30–OK	29–OK	31–OK	30–OK	30–OK	29–OK
S brand								
71RBD	30–OK	29–OK	29–OK	28–OK	30–OK	29–OK	29–OK	28–OK
N brand								
70CBD	30–OK	29–OK	29–CD	28–OK	28–CD	29–ID	29–CD	28–ID
H brand								
63MBD	31–OK	30–OK	30–ID	29–OK	31–OK	30–OK	30–OK	29–OK

TABLE 1 (*(Continued)*

Batch No.—SS Series	SAN GABRIEL SAND MORTARS				20-30 OTTAWA SAND MORTARS			
	7 Day Cure		28 Day Cure		7 Day Cure		28 Day Cure	
	In at Once Wet	Dried 21 Da. First	In at Once Wet	Dried 28 Da. First	In at Once Wet	Dried 21 Da. First	In at Once Wet	Dried 28 Da. First
ORDINARY COMMERCIAL PORTLAND CEMENTS								
R brand								
5V	38–CD	39–CD	29–CD	38–CD	23–CD	40–ID	19–CD	38–CD
58V	19–CD	30–ID	19–CD	28–CD	10–CD	20–CD	8–CD	14–CD
S brand								
57R	20–CD	30–ID	15–CD	22–CD	16–CD	28–CD	8–CD	17–CD
N brand								
55C	13–CD	30–CD	8–CD	18–CD	7–CD	12–CD	7–CD	9–CD
H brand								
56M	9–CD	12–CD	8–CD	9–CD	7–CD	10–CD	7–CD	7–CD
HIGH EARLY STRENGTH CEMENTS								
Y brand One Day								
22Y	35–CD	36–CD	36–OK	35–ID	36–CD	36–ID	36–ID	34–CD
29Y	33–CD	35–ID	35–OK	34–ID	36–ID	35–ID	35–OK	33–CD
48Y	32–CD	34–OK	33–CD	33–ID	35–OK	34–OK	34–OK	33–OK
74Y	29–CD	29–OK	26–CD	28–OK	28–CD	29–OK	29–OK	28–OK
H brand High Early								
49VE	9–CD	22–CD	9–CD	10–CD	9–CD	12–CD	6–CD	10–CD
60VE	10–CD	19–CD	8–CD	17–CD	8–CD	10–CD	7–CD	7–CD
G brand High Early								
51GHE	21–CD	22–CD	11–CD	10–CD	10–CD	12–CD	11–CD	10–CD
52GS	34–ID	33–OK	33–ID	37–CD	33–CD	33–OK	32–CD	30–CD
59G24	13–CD	29–CD	8–CD	18–CD	7–CD	12–CD	7–CD	7–CD
SPECIAL LOW ALUMINATE CEMENTS (Laboratory cements)								
S brand								
72R08	30–ID	29–ID	29–OK	28–OK	28–CD	29–OK	29–OK	28–ID
73R13	30–OK	29–OK	29–OK	28–OK	30–OK	29–OK	29–OK	28–OK
SPECIAL HIGH ALUMINA CEMENTS								
AL brand rapid hardening cement								
45L	35–OK	34–OK	34–OK	33–OK	35–OK	31–CD	34–OK	31–CD
CF brand—a French cement similar to AL								
23F	18–CD	18–CD	12–CD	17–CD	12–CD	12–CD	12–CD	16–CD
35F	23–CD	23–CD	23–CD	20–CD	30–CD	26–ID	29–CD	26–CD
36F	17–CD	22–CD	22–CD	21–CD	33–CD	33–CD	33–CD	26–CD
37F	16–CD	16–CD	16–CD	15–CD	26–CD	26–CD	26–CD	24–CD
38F	20–CD	23–CD	23–CD	24–CD	26–CD	31–CD	27–CD	28–CD

jars were stored in a cabinet at room temperature which was generally between 60° F. and 80° F. For a few hours occasionally on winter nights or summer days, 50° F. and 90° F. may have been reached.

RESISTANCE TO 1 PER CENT SODIUM SULPHATE SOLUTION

Time to disintegration was considered the most significant indicator of sulphate resistance. This is presented in Table 1 for each of the eight mortar treatments for each of the cements in this series.

Table 2 shows the hypothetical compound composition, or description, of the cements mentioned in Table 1. Study of these tables together reveals that in each case where no disintegration is evident,

the C_3A compound is present in an amount usually lower than 8 per cent. Variations in composition of cements which finally fail, although at different periods of exposure, are considered to have relatively little significance. Comparison of results with N brand Boulder Dam cement and *N* brand ordinary commercial portland cement having nearly the same C_3A content (8 per cent and 9 per cent respectively) may be an indication that the lower C_3S content of the former (31 per cent against 55 per cent) contributed to its greatly superior resistance. The results for these periods of exposure (between 2 and 3½ years) in dilute 1 per cent sodium sulphate solutions are considered conclusive only as to the failures. The ultimate and relative resistance of cements showing no susceptibility to corrosion cannot be accurately distinguished or predicted in this period under the conditions of this series of tests. Accelerated tests in stronger solutions and a strength test of the specimens at various half-year or one-year periods are required to make such distinctions.

TABLE 2—HYPOTHETICAL COMPOUND COMPOSITION OF CEMENTS IN TABLE 1

Cement	C_3S	C_2S	C_3A	C_4AF	Sulphate Resistance SS Series Table 1
Portland-pozzolan clinker only					
Z brand High Silica	49	25	14	9	Poor
Z brand High Silica	53	19	7	16	Fair
H brand Tufa	43	32	15	7	Poor
M brand A and B	*N* bran dcommercial clinker ground with silica sand, pozzolan, hydrated lime and colloidal magnesium silicate				Fair
Morris Dam Low Heat					
R brand	49	33	5	7	Good
S brand	26	45	5	15	Good
N brand	20	51	4	17	Good
H brand	33	45	4	13	Good
Boulder Dam Low Heat					
R brand	32	51	5	6	Good
S brand	35	36	3	18	Good
N brand	31	40	8	12	Fair
H brand	17	54	5	16	Good
Ordinary Commercial Portland					
R brand	56	19	9	9	Poor
S brand	49	24	9	7	Very poor
N brand	55	18	9	8	Very poor
H brand	54	20	11	8	Very poor
High Early Strength					
Y brand One Day	57	21	6	7	Good
H brand One Day	70	3	12	7	Very poor
G brand One Day	59	8	11	14	Very poor
G brand One Day	49	30	8	7	Fair
Special Low Aluminate (Laboratory Cements)					
S brand	60	15	1	18	Good
S brand	53	30	2	9	Good
Special High Alumina					
AL brand	Alumina 40%, Lime 37%				Fair
CF brand	Alumina 40%, Lime 37%				Poor

TABLE 3—COMPRESSIVE STRENGTH OF MORTARS AFTER ACCELERATED SODIUM SULPHATE TEST

2 by 4-In. Cylinders of 1:3 Plastic Mortars Using Regraded San Gabriel Sand and W/C by Volume of .95

Batch No. AD	Cement	Type of Exposure													
		70° F. Water				70° F. 10% Sulphate				160° F. Water			160° F. 10% Sulphate		
		3 mo.	6 mo.	1 yr.	2 yr.	3 mo.	6 mo.	1 yr.	2 yr.	3 mo.	6 mo.	1 yr.	3 mo.	6 mo.	1 yr.
4C	*N* brand Commercial	4355	4780	5033	....	3480	6 mo.	....	....	4930	5025	4747	5057	3666	4020
5M	*H* brand Commercial	4535	4903	5070	....	2970	6 mo.	....	....	4988	5138	5733	5507	5915	5323
6R	*S* brand Commercial	3690	4350	4875	....	3853	6 mo.	....	....	4427	4843	4877	5135	2333	3333
7V	*R* brand Commercial	4490	5030	5440	....	3253	6 mo.	....	....	4710	5045	5967	5800	5727	5817
19C	*N* brand Boulder Dam	4162	5030	5320	....	4240	2425	12 mo.	....	4158	4253	5957	4435	4763	5547
12M	*H* brand Boulder Dam	3837	5505	5733	....	3565	4320	12 mo.	....	3890	4127	6167	3737	5043	7123
20R	*S* brand Boulder Dam	4563	5230	5607	5002	4695	3677	4070	3040	4983	4703	....	5555	5943	6987
11V	*R* brand Boulder Dam	4020	5675	6590	5400	4077	5350	6050	3327	4610	5480	6320	5533	6512	7123
9VE	*H* High Early	4903	5595	5557	....	2400	6 mo.	....	....	4637	4813	4700	5547	1290	10 mo.
8G	*G* High Early	4583	4993	5092	....	3272	6 mo.	....	....	4297	4648	5523	4815	1803	1860
22Y	*Y* High Early	6150	6330	6407	....	4580	1673	12 mo.	....	5973	5597	5290	5625	6023	6543
1S	*Z* Composite	5587	5765	5867	....	5517	5363	12 mo.	....	5150	5300	6123	5280	5930	5897
2S	*Z* Std. Clinker	6255	6278	5900	....	5720	5880	12 mo.	....	5542	5275	5613	5857	6122	6243
3S	*Z* Low C_3A Clinker	5600	5870	5680	5247	5845	5927	5017	2647	5003	5118	5937	5318	5437	5223
10T	*H* brand Tufa	3797	4420	4730	....	3320	3008	11 mo.	....	3535	4172	5780	3298	2435	2103
24R	*S* brand Special 2408	6058	5493	7077	6660	6138	5743	5063	5600	5980	6433	6273	6488	6823	7553
25R	*S* brand Special 2413	6220	6373	7060	6969	6498	6666	7460	6713	6130	6387	7360	5922	7357	7997

Note: Each value average of 3 specimens. 28-Day moist cure plus 2, 5, 11 or 23 months' exposure as above, then tested wet.
Values in months denote age at time of complete disintegration.
For graphical presentation of data in this Table see Fig. 1 following.

OUTLINE OF LATER AND ACCELERATED TESTS

Throughout the early tests, 1 per cent sodium sulphate solutions were used in preference to stronger solutions because they were doubtless more nearly representative of soil water to be encountered, and it was then assumed that there was sufficient time to draw safe conclusions as to the ultimate or relative resistance of all the cement compositions investigated. As more resistant compositions included in the early tests continued unaffected in the 1 per cent solution, increasing doubt was placed on the assumption that there would be time for the slower test to make conclusive distinctions.

Accelerated sodium sulphate action

For the above reason, late in 1933 and early in 1934 a series (AD) of regraded San Gabriel sand mortars was made, using what then promised to be the most representative of the cements used in the 1 per cent sulphate test in the SS series. Special features of this test were (1) acceleration of the sulphate action by means of a stronger solution (10 per cent) and (supposedly) by a higher temperature (160° F.); and (2) measurement of results by comparison of the compressive strength of 2 by 4-in. mortar cylinders tested wet after various periods of sulphate solution exposure with that of similar specimens immersed in plain water at the same temperature for the same period. These specimens were made for testing at 3 months, 6 months, 1 year and 2 years of storage in water at 70° F., 10 per cent sodium sulphate solution at 70° F., water at 160° F. and 10 per cent sodium sulphate solution at 160° F.

TABLE 4—HYPOTHETICAL COMPOUND COMPOSITION OF CEMENTS IN TABLE 3

Cement	C_3S	C_2S	C_3A	C_4AF	Sulphate Resistance AD Series Table 3
N brand ordinary comm.	55	18	9	8	Very poor
H brand ordinary comm.	54	20	11	8	Very poor
S brand ordinary comm.	49	24	9	7	Very poor
R brand ordinary comm.	56	19	9	9	Very poor
N brand Boulder Dam	31	40	8	12	Poor
H brand Boulder Dam	17	54	5	16	Poor
S brand Boulder Dam	35	36	3	18	Fair
R brand Boulder Dam	32	51	5	6	Good
H brand High Early	70	3	12	7	Very poor
C brand High Early	59	8	11	14	Very poor
Y brand High Early	57	21	6	7	Poor
Z brand composite samples 1 to 18, SS series					Poor
Z brand ordinary clinker	*49	25	14	9	Poor
Z brand low C_3A clinker	*53	19	7	16	Fair
H brand Tufa	*43	32	15	7	Poor
S brand Special 2408	60	15	1	18	Good
S brand Special 2413	53	30	2	9	Excellent

*Portland-pozzolan cements, composition is of clinker only.

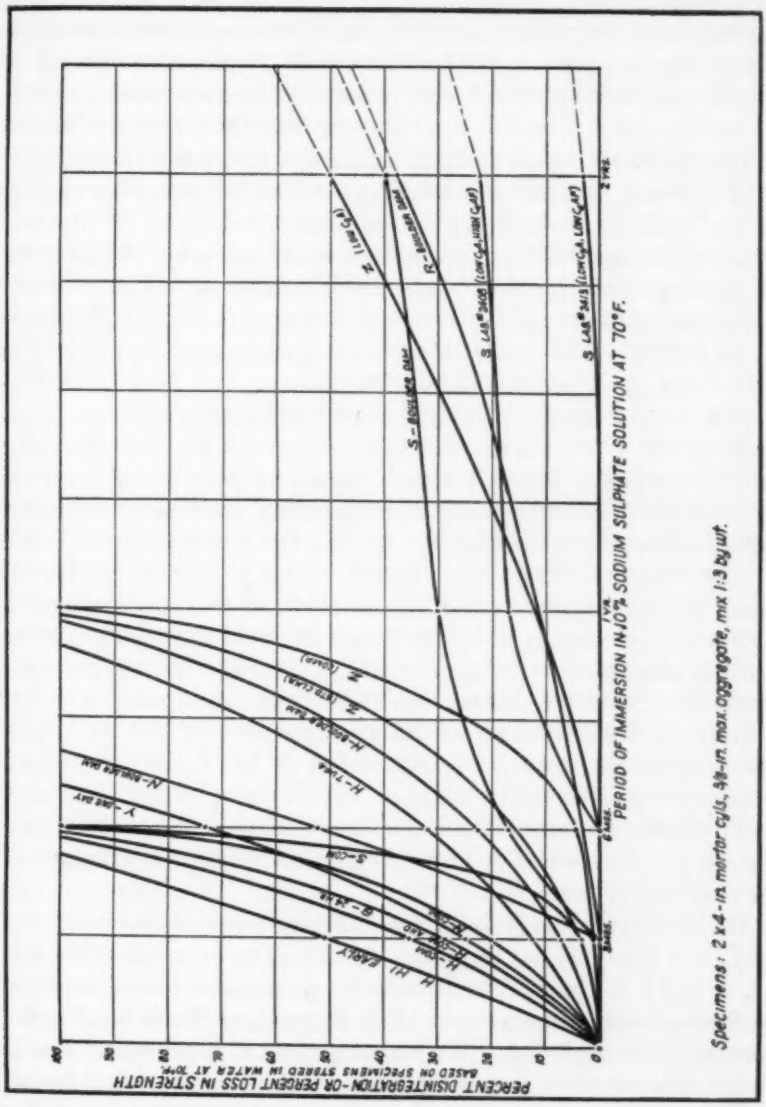

PERCENT DISINTEGRATION-OR PERCENT LOSS IN STRENGTH
BASED ON SPECIMENS STORED IN WATER AT 70°F.
PERIOD OF IMMERSION IN 10% SODIUM SULPHATE SOLUTION AT 70°F.
Specimens: 2 x 4-in. mortar cyls, 3/4-in. max. aggregate, mix 1:3 by wt.

FIG. 1—RESISTANCE OF VARIOUS CEMENTS TO DISINTEGRATION BY SODIUM SULPHATE (AD SERIES)

Results of the accelerated disintegration tests are reported in Table 3, and shown graphically in Fig. 1 as the percentage of the compressive strength of the mortars lost after various periods of exposure in 10 per cent sodium sulphate solution at 70° F. In Table 4 may be found the hypothetical compound composition of the cements included in the tests, together with a word indicating its degree of resistance to the accelerated test in 10 per cent sodium sulphate solution. From Fig. 1 and Tables 2, 3 and 4, the following conclusions are evident:

1. The test in 10 per cent sodium sulphate solution is a true acceleration of the tests in 1 per cent solutions, in that no anomalies appear when the relative effects of the two are compared. The results also substantiate the generally accepted conclusion that, other things being equal, the resistance of a cement is closely related to its C_3A content.

2. Increasing the temperature of the 10 per cent solution test from 70° F. to 160° F. in no case accelerated the sulphate action. In fact, in many cases the effect was quite the contrary.

3. The *S* brand special composition 2413 is conspicuously the most resistant of all. Comparing the exposure at 70° F., it is the only one which showed a higher strength after one year in 10 per cent sodium sulphate solution than corresponding specimens in moist or underwater cure, as well as higher than the 6-month tests of the same cement in the 10 per cent solution. This is believed due to an extremely low percentage of C_2A (2 per cent), in combination with a relatively low percentage of C_4AF (9 per cent), and possibly due to the normal proportions of C_3S and C_2S (53 per cent and 30 per cent respectively). Basis for this conclusion lies in the composition of the *S* brand special 2408 which shows definite signs of decline in its 1- and 2-year strength in the 10 per cent solution at 70° F., in comparison with the specimens stored in water at 70° F. (see Fig. 1). Although this composition contained 1 per cent less C_3A than 2413, C_4AF was high at 18 per cent and C_3S and C_2S were more disproportionate at 60 per cent and 15 per cent.

4. Not readily explainable is the difference in resistance of the Boulder Dam cements, but the cause is believed to lie in the difference in C_3A or C_4AF content. The *R* brand cement is considered superior to the *S* brand cement because C_4AF is 12 per cent less in the former at 6 per cent, although C_3A is 2 per cent greater at 5 per cent. For *R* brand the sum of these two compounds is 11 per cent, for *S* brand 21 per cent, thereby allowing 10 per cent more of the active and desirable C_3S and C_2S compounds in the former, which apparently caused its superior strength and apparently greater resistance in the 10 per cent sodium sulphate solution.

5. Less readily explainable is the difference in results with *H* brand and *S* brand Boulder Dam cements. The superior results with the *S* brand cement are believed due to proportionately much lower C_3A (3 per cent as compared with 5 per cent in the *H* brand cement), and to the more nearly normal balance between the C_3S and C_2S compounds in the *S* brand cement, which was found to be 35 per cent and 36 per cent as compared with 17 per cent and 54 per cent respectively in the *H* brand cement composition.

6. The *N* brand Boulder Dam cement failed apparently because its percentage of C_3A was much higher than any of the others (8 per cent).

7. It should be emphasized that in these results with this group of varied cement composition, there is significant evidence that compounds other than C_3A have an important influence upon the resistance of the cement to sulphate solutions such as C_4AF particularly, and probably C_3S.

EXTENSION OF THE ACCELERATED TESTS

The purpose of the X series of tests was to extend the accelerated test series AD in conditions of exposure, in the number of different cement compositions included, and to larger specimens of concrete in place of mortar specimens. In the program of tests, the 160° F. immersion of the AD series was eliminated, and partial immersion in 1 per cent solution outdoors was added. The complete program of specimens and tests of each cement is shown in Table 5. The program was arranged for test of 11 special laboratory cements which were under general investigation, several special portland-pozzolan commercial cements submitted as being relatively sulphate resistant, and the four local brands of ordinary portland cement for comparison. Also included were 43 portland-pozzolan cements prepared by the University of California in a cooperative test program of such cements. Results of tests on this latter group are not included in this paper but may be found in the paper, "Properties of Mortars and Concretes Containing Portland-Pozzolan Cements," by Davis, Kelley, Troxell and Davis.*

Preparation and treatment of specimens.

For the 3 by 6-in. cylinders used for a compression test of the effect of sodium sulphate, a concrete with an aggregate of ¾-in. maximum size and a cement content of 1.50 bbl. per cu. yd. was used for a close parallel to job concrete. In these concretes, proportions of cement to aggregate were maintained constant by weight rather than by volume, as this usually gives most directly comparable results regardless of

*JOURNAL, American Concrete Institute, Sept.-Oct., 1935; *Proceedings* Vol. 32, p. 80.

TABLE 5—X SERIES, PROGRAM OF TESTS

Investigation to determine relative resistance of various compositions of Portland and Portland-pozzolan cements in mortars and concretes to disintegration by sodium sulphate solutions under various conditions.

No. and Size Specimen	Mix	Water	Consistency	Aggregate	Test	Exposure Condition	Exposure Temperature	Time Exposed After 28 Day Cure Before Test
8—2" x 6"	Concrete 1.5 bbl. c. y. by weight	Variable	Constant—2½" flow	San Gabriel	Compression	–10% Na_2SO_4	70° F.	6 mo. and 1 yr.
8—3" x 6"						– 1% Na_2SO_4	70° F.	6 mo. and 1 yr.
8—3" x 6"						–Water	70° F.	6 mo. and 1 yr.
8—3" x 6"						– 1% Na_2SO_4	Outdoors	6 mo. and 1 yr.
8—3" x 6"						–Water	Outdoors	6 mo. and 1 yr.
8—3" x 6"						–Moist cure	70° F.	6 mo. and 1 yr.
2–2x2x18"	Mortar 1:4 by Weight	Constant—w/cv — 1.10	Variable with cement	Ottawa	Volume chang. change	–10% Na_2SO_4	70° F.	Test continuous No cure. Placed in solution at 3 days on removal from molds.
1–2x2x18"						–Water	70° F.	

In addition to the above general program for all cements, the same was repeated for five of the laboratory cements (D13, D18, E13, E18, and F18) with the difference that the first seven days of the 28-day moist curing were in the moist steam room at 130° F. Volume change bars in these groups were steam cured at 130° F. first 3 days only.

differences in specific gravity of the cements. The water content was varied as necessary with each cement to produce a uniform consistency as measured by a 2½-in. flow in the dry 6-in. Burmister flow trough with 10 one-inch drops. The aggregate was washed sand and gravel from the San Gabriel wash, screened and regarded as follows:

Sieves:	0 — 50	— 30	— 16	— 8	— 4	— ⅜"	— ¾"
Per cent each:	9	14.4	11.4	8.4	8.4	8.4	40

This grading has a fineness modulus of 4.75. Four concrete specimens were made for each test period in each condition of curing or sulphate exposure. All specimens were vibrated on a small magnetic vibrator and were cast in 3 by 6-in. cylindrical cardboard molds with integral newsprint lining dipped in paraffin, and with an oiled plate glass disc placed in the bottom of each to eliminate necessity for capping the lower end of each specimen. All of these specimens were cured 28 days in the moist room at 70° F. before introduction to the exposure designated in Table 5.

The 2 by 2 by 18-in. mortar bars were made and measured for expansion by an optical comparator to determine whether expansion of such bars in 10 per cent sodium sulphate solutions at reasonably early ages could be used as suggested by Thorvaldson as an indicative test of the ultimate resistance of the cement to disintegration by sodium sulphate solutions. A lean plastic mortar was used composed of 1:4 proportions by weight with 20-30 Ottawa sand and a w/c by volume of 1.10. The bars were placed in the 10 per cent solution and in the water, immediately after removal of the forms at the age of 3 days.

The sodium sulphate solutions were made up using Glauber's salt, to the specific gravity necessary for the desired strength. The percentage indicated is the percentage by weight of anhydrous sodium sulphate. General experience with such solutions in concrete tests indicated the desirability of correcting the solution by adding sulphuric acid until the solution was neutral to the phenolphthalein indicator, and this was done at 2-week intervals. At this time the solutions were checked for sodium sulphate content and sulphate was added when dilution had occurred.

The outdoor specimens were placed in pans exposed to the weather. The pans were filled with water or with the 1 per cent solution as scheduled to 1½ in. below the top of the cylinders, and neither water nor solution was changed or altered until it had evaporated to a point 1½ in. from the bottom. Then all the pans were again filled to the former depth with water and the 1 per cent solution was corrected

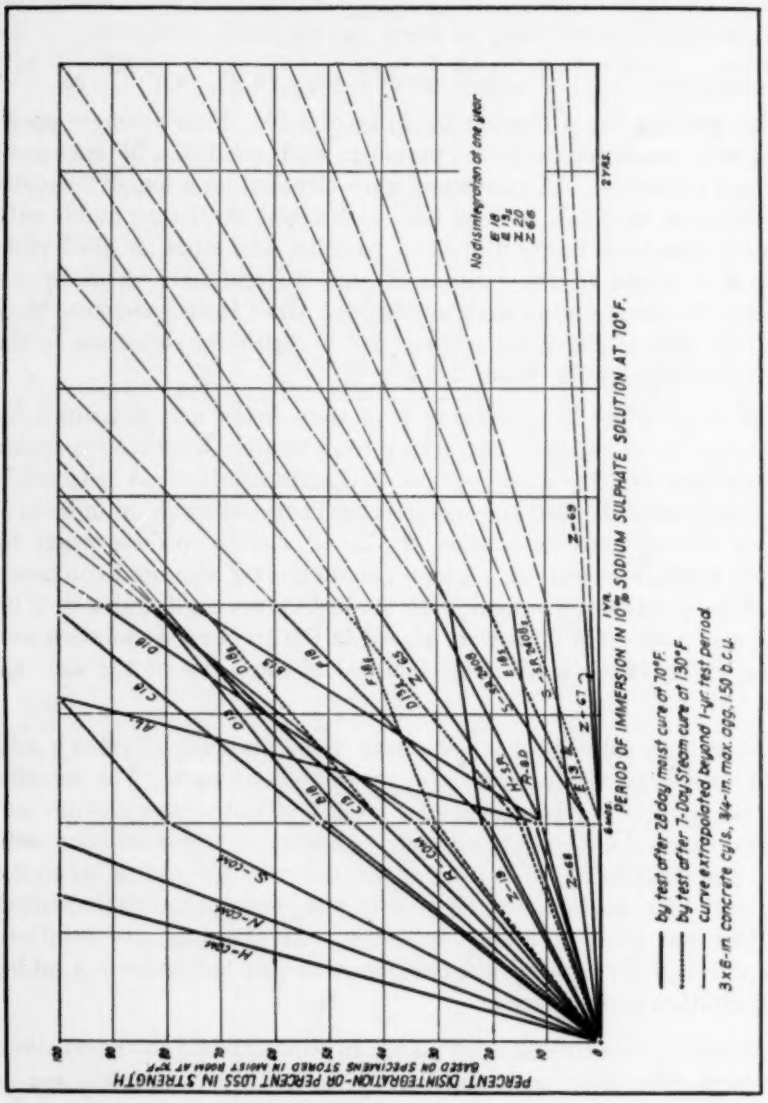

PERCENT DISINTEGRATION—OR PERCENT LOSS IN STRENGTH
BASED ON SPECIMENS STORED IN MOIST ROOM AT 70°F
PERIOD OF IMMERSION IN 10% SODIUM SULPHATE SOLUTION AT 70°F.
No disintegration at one year
by test after 28 day moist cure at 70°F.
by test after 7-Day Steam cure at 130°F
curve extrapolated beyond 1-yr test period
3x6-in. concrete cyls., ¾-in. max. agg., 150 b.c.u.

FIG. 2—RESISTANCE OF VARIOUS CEMENTS TO DISINTEGRATION BY SODIUM SULPHATE (X SERIES)

with sulphuric acid as described above. This test was not as uniform as the 70° F. tests because all the specimens could not be placed outdoors at the same time, with the result that each group was exposed under a somewhat different weathering history. It was noted that specimens which failed in this test disintegrated first above the water line. This indicated that, where evaporation worked in conjunction with the salt, the mechanical disruption caused by crystallization expansion aided the chemical corrosion of the mortar. This form of corrosion was more evident on the specimens containing the more coarsely ground of two cements of similar composition.

Results of the compression tests are arranged in Fig. 2 and Tables 6 and 7, which are largely self-explanatory. Specimens were tested wet as removed from the solutions or moist cure. Although the results clearly corroborate the lack of resistance of certain cement compositions similarly demonstrated in previous test series, the differences in the more resistant cements are not as clearly shown as would have been accomplished by similar compression tests following considerably longer periods of storage in 10 per cent sodium sulphate solutions. To distinguish more clearly among the cements which showed up well, marked as "good" in Table 7, specimens stored in the 1 per cent solution for one year were not broken at that time, but were transferred to the 10 per cent solution for at least another year or until their relative resistances are clearly demonstrated. The results of these supplementary tests must necessarily be reported at a later date.

Table 6 and in Fig. 2 show the effect of the various sodium sulphate storage conditions upon the 3 by 6-in. concrete specimens. The parallelism of results of accelerated tests in the 10 per cent solutions with results after a longer time in weaker solutions has been generally demonstrated in the *SS* and *AD* series. (Compare Tables 2 and 4). Hence it is considered that the most significant values in Table 6 are in the right hand column, which show the strength in the 10 per cent solution expressed as a percentage of the strength of the moist-cured specimens. When this percentage was over 85 per cent the cement was listed as "good" in Table 7 and specimens which survived one year in the 1 per cent solution were transferred without test to the 10 per cent solution for at least another year as explained above.

In Table 6 is also recorded the difference in expansion of the lean Ottawa sand mortar bars stored in 10 per cent sodium sulphate solution and of similar bars stored in plain water. This greater expansion of the bars in the sodium sulphate solution is shown in millionths of an inch per inch at the end of 60 and 120 days where the bar has not dis-

TABLE 6—EXPANSION AND COMPRESSIVE STRENGTH OF MORTARS AND CONCRETES AFTER STORAGE IN SODIUM SULPHATE SOLUTIONS

X Series Batch Number	1:4 Ottawa Mortar Expansion Greater than That in Water in Millionths		¾-In. Max. Size Concrete, 1.50 bbl. per cu. yd.—3 by 6-in. Specimens												% Resistance in 10% Solution Moist Cure Basis 1 yr.
			In Water				In 1% Na_2SO_4				Moist Cure		10% Na_2SO_4		
			at 70° F.		Weather		at 70° F.		Weather		at 70° F.		at 70° F.		
	60 Day	120 Day	6 Mo.	1 Yr.	6 Mo.	1 Yr.	6 Mo.	1 Yr.	6 Mo.	1 Yr.	6 Mo.	1 Yr.	6 Mo.	1 Yr.	
Laboratory cements made at University of California:															
1	880	Disint.	3825	4202	3937	4232	3700	2040	2642	11.5 mo.	3990	4157	5.5 mo.	....	0
2	600	Disint.	5122	4872	5122	5320	5010	2877	4228	4255	5557	5600	5.5 mo.	....	0
3	160	370	3560	4052	3370	4280	3492	3910	3195	3270	3597	3967	2035	933	24
4	150	720	4760	4925	4730	5185	4642	4347	4572	4675	4787	4980	2600	946	19
5	160	510	3601	4015	3279	3920	3689	3575	3460	4110	3383	3675	2028	628	17
6	190	1370	5610	5740	5285	5695	5260	4750	5315	5415	5054	5050	2406	11.8 mo.	0
7	290	520	3838	4075	3809	4440	3826	3930	3555	4215	3813	3950	1856	680	17
8	240	1400	5755	5445	5748	6030	5958	5365	6033	5880	5435	5825	3668	495	8
9	200	210	3388	4190	3203	3975	3693	*	3318	....	3423	4675	3823	4030	86
10	160	200	5402	5035	4530	5255	5132	*	5140	....	4803	5340	5182	5440	101
11	170	230	5445	5265	5370	5580	5420	5780	5155	5230	4453	4835	3515	1930	40
Some of above and other cements steam-cured at 130° F. first 7 days:															
13	380	1020	3175	3495	2947	3324	3407	3490	3360	3445	3592	3600	3066	1585	52
14	750	Disint.	5307	4960	5055	5260	5150	5173	5142	5330	5115	5465	3057	1410	26
15	170	200	2630	2820	2605	2835	2802	3075	2847	2847	2632	2745	2860	3003	109
16	170	230	4107	4550	4090	4500	4030	4320	4147	4600	4187	4510	4125	3580	79
17	260	Disint.	3822	4480	4325	4855	4212	4500	4452	4695	4400	4430	3032	2046	46
18	80	160	3875	4820	4175	4280	4035	*	4175	....	3935	4745	4135	4093	86
Special cements submitted as being particularly resistant to Na_2SO_4:															
12	140	180	4357	4270	4232	4530	4330	4155	4502	4675	4302	4550	4207	3290	72
26	190	530	4183	4640	4120	4640	4908	5000	4590	4520	4695	5015	4180	3447	69
70	not made		5412	5700	5152	5465	5410	5450	5012	3300	5640	5375	5012	4502	84
25	(spalling—48 da. without expansion)		1668	1585	3260	2010	1923	1805	3055	11.5 mo.	1883	2070	1650	9.5 mo.	0
Ordinary commercial Portland cements:															
21	890	Disint.	3902	3770	3707	3735	3667	3145	3852	3470	3845	3965	5.5 mo.	....	0
22	720	Disint.	3647	3500	3482	3655	3140	11.5 mo.	3400	2900	3592	3190	4.0 mo.	....	0
23	690	Disint.	3770	4380	3995	4190	3760	2385	3830	4120	4007	3995	815	9.6 mo.	0
24	280	850	4260	4600	4200	4610	4065	4285	4580	4825	4392	4825	2897	1030	16
Z brand high silica cements:															
19	160	220	5305	5635	5352	5390	4880	5170	5060	5105	5622	5560	4377	3990	72
20	130	200	5455	4875	5274	5000	5130	*	4840	....	4987	4940	5017	5037	102
66	80	100	4330	4300	4707	4945	4622	5120	4550	4380	4735	4835	4655	4905	101
67	100	110	4807	4925	5100	5350	5147	5410	4897	5385	5335	5395	5510	5300	98
68	30	30	4402	4570	4060	4370	3942	4435	4307	3530	4282	4575	4492	4287	94
69	50	50	4890	5175	4895	5295	4875	*	4640	5500	4927	5190	4970	5048	97
65	210	800	4422	4950	4722	4800	4537	3380	4772	5530	4717	4485	4052	2592	58

Values are averages of 4 specimens. *Transferred to 10 per cent solution.

TABLE 7—HYPOTHETICAL COMPOUND COMPOSITION AND OTHER PROPERTIES OF CEMENTS IN X SERIES—TABLES 6 AND FIG. 2

X Series Batch Number	Cement	% Blending Material	Fineness Spec. Surf. by Micr.	Specific Gravity	C_3S	C_2S	C_3A	C_4AF	Sulphate Resistance Table 6
Laboratory cements made at University of California:									
1	A1300	0	1310	3.14	53.0	23.4	12.1	7.6	very poor
2	A1800	0	1830	3.14	53.0	23.4	12.1	7.6	very poor
3	B1300	0	1290	3.14	60.5	14.5	7.9	12.6	poor
4	B1800	0	1830	3.14	60.5	14.5	7.9	12.6	poor
5	C1300	0	1320	3.14	53.8	23.0	8.7	11.0	poor
6	C1800	0	1830	3.14	53.8	23.0	8.7	11.0	very poor
7	D1300	0	1310	3.14	40.5	33.9	7.8	13.6	poor
8	D1800	0	1840	3.14	40.5	33.9	7.8	13.6	very poor
9	E1300	20	1325	3.01	60.5	14.5	7.8	12.6	good
10	E1800	20	1830	3.01	60.5	14.5	7.9	12.6	good
11	F1300	10	1910	3.08	40.5	33.9	7.8	13.6	poor
Some of above and other cements steam-cured at 130° F. first 7 days:									
13	D1300	0	1310	3.14	40.5	33.9	7.8	13.6	poor
14	D1800	0	1840	3.14	40.5	33.9	7.8	13.6	poor
15	E1300	20	1325	3.01	60.5	14.5	7.9	12.6	good
16	E1800	20	1830	3.01	60.5	14.5	7.9	12.6	fair
17	F1800	10	1910	3.08	40.5	33.9	7.8	13.6	poor
18	*S* brand sulphate resistant	0	1580	3.14	63.0	13.0	2.0	16.0	good
Special cements submitted as being particularly resistant to Na_2SO_4:									
12	*S* brand sulphate resistant No. 2408	0	1580	3.14	63.0	13.0	2.0	16.0	fair
26	*H* brand sulphate resistant	0	1570	3.14	22.0	52.5	1.4	18.1	fair
70	*R* brand Boulder Dam	0	1565	3.17	35.4	46.7	4.9	6.4	good
25	*AL* brand early hardening	0	1410	3.14	40% alumina, 37% lime				very poor
Ordinary commercial portland cements:									
21	*N* brand	0	93/200	3.14	47.1	24.8	10.7	7.8	very poor
22	*H* brand	0	91/200	3.14	42.0	31.7	11.0	8.9	very poor
23	*S* brand	0	97/200	3.14	49.8	24.1	9.4	7.2	very poor
24	*R* brand	0	96/200	3.14	61.6	14.1	6.6	8.2	poor
Z brand high silica cements—blended with:									
19	80% diat. shale, 20% lime	30	97/200	2.88	48.7	25.6	13.9	2.0	fair
20	80% diat. shale, 20% lime	30	98/200	2.90	53.0	19.4	7.1	16.3	good
66	90% diat. shale, 10% lime approx.	30	97/200	2.84	55.1	17.6	5.8	16.2	good
67	90% diat. shale, 10% lime approx.	15	95/200	2.99	55.1	17.6	5.8	16.2	good
68	100% diat. shale, no lime	30	98/200	2.82	55.1	17.6	5.8	16.2	good
69	100% diat. shale, no lime	15	98/200	2.99	55.1	17.6	5.8	16.2	good
65	*Z* brand unblended	30	97/200	3.08	55.1	17.6	5.8	16.2	fair

integrated prior to one or the other of these ages. Study of these expansions in comparison with results of more conclusive measures of sulphate resistance shown in the right hand column of Tables 6 and 7 reveals that this method of evaluation of relative resistance of cements to disintegration by sodium sulphate is much less reliable and distinctive than the compression test method, and that it is definitely unreliable in comparing cements of different types such as portland-pozzolan with unblended cements.

Table 7 lists the hypothetical compound composition and other properties of the cements investigated in the *X* series. In the case of portland-pozzolan cements the compound composition is of the clinker only, the percentage of blending material being noted in a separate column.

It will be noted in Table 7 that the group of laboratory cements includes two carefully controlled degrees of fineness for each composition. Although somewhat less resistant than the 1300 fineness cement in the indoor tests, the finer cement (1800 fineness) was generally superior in the outdoor tests. In neither case was the superiority of one or the other sufficient to produce ultimate resistance.

Aside from the superior resistance of the portland-pozzolan cements in this series as compared with unblended portland cements of the same clinker, most promising results to date were obtained with the *R* brand Boulder Dam cement which contained less than 5 per cent C_3A and less than 7 per cent C_4AF (see Fig. 2). It will be recalled that in the AD series, compositions having minimum percentages of both these compounds showed the least susceptibility to the sulphate.

In connection with the superiority of portland-pozzolan cements in this series, a note of caution is sounded, for among many other such cements investigated but reported elsewhere, there were many failures. Study of pozzolanic and other blending materials is not a part of this investigation but such a study has been made by the University of California and reported in connection with the tests of the other portland-pozzolan cements previously mentioned. It can only be stated here that where such a cement is under consideration, the blending materials and their preparation should be carefully selected and designated. Further, the clinker used as a basis for the blended cement should have a low C_3A content. (Compare the effects of clinker composition in the *Z* brand blended cements—Tables 6 and 7). In other words, no admixture should be depended upon to correct the defects of an unresistant cement.)

ACCEPTANCE TESTS FOR SULPHATE RESISTANT CEMENT

Purpose of such acceptance tests

In the purchase of any material for a specific purpose, it is the usual procedure to require that the material demonstrate its suitability for that purpose by means of some form of actual or indicative performance tests. Because acceptance or rejection of the material is often based on such tests, they are commonly called acceptance tests.

Thus in the purchase of sulphate resistant cement, it is particularly desirable to know before using the cement that it is definitely resistant to corrosion in sodium sulphate waters to the required degree. To be practicable, such a test must show definitely conclusive and dependable results in a relatively short time—preferably within 28 days after making the test specimens from the cement sample.

The tests of mortars and concretes already described required too long a time to meet this requirement, and more accelerated tests were sought. Tests on small neat cement slabs which were being used in current cement specifications were tried in this laboratory, but were found to be insufficiently sensitive during the 28-day period to make definite differentiation between cements of good and those of mediocre sulphate resistance. The test was therefore considered to be entirely inadequate to insure receipt of cement of the degree of sulphate resistance desired and known to be available. The extensive tests herein reported had already been made, and from the results it was known that certain compositions of portland cement, which could be determined by ordinary chemical analysis, were consistently resistant to sulphate corrosion to a very high degree. It was therefore decided that most dependable results would be assured by specification of a cement having a composition known to be effective. Acceptance tests then involved only routine chemical analysis to determine if the composition fell within the required limits. Long time tests in 10 per cent sodium sulphate solutions have shown that cement thus accepted has reasonably well flulfilled all expectations. (Table 9).

Although in this case satisfactory results were obtained in the above manner, it was recognized that a dependable 1-month acceptance test would be of great value, particularly under the following conditions: (1) where little preliminary investigation was possible; (2) where facilities were not available for the complete and accurate chemical analysis necessary for the computation of hypothetical compound composition; (3) as a check on the chemical test acceptance method mentioned above; and (4) as a quick method of selecting cements having sufficient merit to warrant their further study and investigation. For these purposes the slab-warping test was developed,

incidental to the District's acceptance tests of sulphate resistant cement. The test has been described in *Industrial and Engineering Chemistry* (*Anal. Ed.*), July, 1936, and is only briefly outlined below.

The slab-warping test which finally evolved from experimentation consisted in the preparation of neat cement slabs having dimensions of 2 x 4½ x ⅛ in. one face being coated with a water-repellent paint and the other carefully scraped to provide a fresh surface for attack. Slabs are cured under water for 7 days, then immersed in a 10 per cent sodium solution for the remainder of the 28-day period. Due to exposure of only one face, expansion resulting from the sulphate attack causes the slab to warp, and the extent of the attack is reflected in the degree of warping.

A spherometer was devised for measuring the extent of the warping, and it was found that the *rate* of the warping rather than the absolute magnitude was the best measure of the susceptibility of the cement to the sulphate action. Accordingly, the difference between the 21-day and the 7-day readings is called the warping index and is used as the basis of comparison.

The results shown in Table 8 for a typical series of portland cements indicate the general validity of the rule that the more resistant the cement the less the expansion and therefore the less the warping.

TABLE 8—CORRELATION BETWEEN SLAB-WARPING AND COMPRESSION TESTS FOR SULPHATE RESISTANCE

Reference Number	Cement—Kind	Slab Warping —1000ths In. of from 7th to 21st Day	% Disintegration Concrete Cylinders After 6 Months in 10% Solution
R	*S* brand Commercial	48	100
A8	Lab. Cement A18	32	100
30C	*N* brand, Specification 79	21½	100
B8	Lab. Cement B18	20	46
C8	Lab. Cement C18	20	52
D8	Lab. Cement D18	17	33
14R	*S* brand Specification 79	16	21
R53	*S* brand Low C.A.	9	2
6V	*R* brand Specification 79	4	0
9V	*R* brand Sulph. Resistant	2	0

The correlation between the results of this test and those of compression in the case of portland-pozzolan cements is not so well defined, but it must be kept in mind that the expansion test has a value as such since abnormal expansion of a structural member may lead to failure even though the strength has not been greatly impaired. For such cements, then, the test may serve as an adjunct to compression or tensile tests.

TABLE 9—RESISTANCE OF METROPOLITAN WATER DISTRICT CEMENTS TO 10% SODIUM SULPHATE SOLUTIONS

Cement Brand	Lot No.	Specific Surface	% C_3A	% C_4AF	Compressive Strength 2 x 4" Cyl.—p.s.i.						% Resistance—Water Storage Basis			Warping Index of Neat Cement Slabs in 10% Na_2SO_4 Typical Values
					In Tap Water at 70° F.			In 10% Na_2SO_4 at 70° F.						
					3 Mo.	6 Mo.	1 Yr.	3 Mo.	6 Mo.	1 Yr.	3 Mo.	6 Mo.	1 Yr.	
Made under M. W. D. Spec. 68—same as A. S. T. M. Spec. C9-30 except fineness over 1600 sq. cm./gr.														
N	6	1670	10.0	8.1	3607	3918	3282	Disint. at 3 mo.			0	0	0	29
H	4	1720	11.8	7.9	4622	4977	....	Disint. at 3 mo.			0	0	0	..
S	13	1850	10.4	6.6	3857	5267	4560	Disint. at 3 mo.			0	0	0	..
R	2	1760	10.0	8.4	3663	5188	4680	Disint. at 3 mo.			0	0	0	20
													0	
								Average 1 yr. resistance in 10% sodium sulphate						
Made under M. W. D. Specification No. 79.*														
N	14	2140	7.1	12.1	4047	4900	....	3792	Dis.	6 mo.	94	0	0	19
H	3	2000+—	6.8	15.7	47.13	5480	4207	4260	431	7 mo.	90	8	0	25
S	26 & 27	1900+—	6.5	13.3	3925	4660	4660	....	2350	1320	....	50	28	16
S	14	1930	6.0	13.2	4548	4880	3566	4188	3323	1193	92	68	33	16
R	3	2100+—	5.7	6.4	5633	6003	4093	5207	5613	2213	93	93	54	10
R	6	2030	5.7	5.6	5420	5457	3590	5087	5163	2950	94	95	82	5
													25%	
								Average 1 yr. resistance in 10% sodium sulphate						
Made under M. W. D. Specification No. 79 modified as described in par. 6 Author's Closure. All are R brand Sulphate Resistant cement:														
R	8	2150	3.7	5.1	3960	5057	5193	4290	4993	5170	108	99	100	..
R	9	1990	3.8	5.0	4318	....	3440	4300	....	2350	100	....	68	2
R	12	2075	3.5	4.9	....	....	4263	....	....	3828	....	....	90	..
R	16 & 17	2200+—	3.5	4.5	....	....	2840	....	....	2820	....	....	99	..
R	22	2143	3.5	4.1	3920	....	4880	3586	3720	3790	92	....	78	..
													87%	
								Average 1 yr. resistance in 10% sodium sulphate						

*C_3S not more than 55%—C_3A not more than 8%—Fineness over 1800 sq. cm. per gm. (Wag. turb.).

COMMENTS AND CONCLUSIONS

From the foregoing investigations, it is believed that the following conclusions are indicated regarding the resistance of concretes to the corrosive action of sodium sulphate solutions:

1. Resistance to sulphate corrosion is evidently very sensitive to the influence of the chemical composition of the cement, and the basis for the preparation of concretes of high resistance is primarily the choice of cement of proper compound composition.

2. Ordinary commercial portland cement *of brands tested*, without exception, is subject to early disintegration in sulphate waters, and no dependence should be placed upon it under such conditions unless thorough tests have proved the contrary. See Tables 1, 3, 6 and 7.

3. Failure of ordinary and high early strength portland cement is apparently due largely to amounts of C_3A above 6 per cent or to disproportionate amounts of C_3S and C_2S. See Tables 2, 4 and 7.

4. Ultimate resistance is questionable when C_3A exceeds 5 per cent, and with C_3A under this limit, percentages of other compounds must be properly adjusted or failure is possible. See Tables 2, 4, 7 and 9.

5. There is significant evidence in the results of this investigation that compounds other than C_3A (particularly C_4AF) have an important influence upon the resistance of the cement to sodium sulphate solutions.

6. The results shown in Tables 3 and 4, supported by check tests, indicate that for a cement dependably resistant to sulphate corrosion the following composition limitations are necessary:

C_3A, not more than	4%
C_3A, plus C_4AF, not more than	12%
C_3S, not more than	50%
C_2S, not less than	30%
7 day tensile strength, standard briquets, not less than	225 p.s.i.
Specific surface (turbidimeter), not less than	2000 sq. cm. per gr.

(This fineness, although desirable for other reasons, is necessary to insure adequate early strength with the above composition).

These are the general characteristics of sulphate resistant cement being purchased by the Metropolitan Water District of Southern California for use where such a cement is required in the Colorado River Aqueduct construction.

7. Great resistance may be obtained by use of a portland-pozzolan cement, as evidenced by the resistance of cements 10, 20, 66, 67, 68 and 69, Tables 6 and 7. However, other portland-pozzolan cements in the cooperative investigation portion of the *X* series mentioned above were complete failures as sulphate resistant cements. Although

the features of the pozzolanic cements, and of materials blended with cement clinker in making them, are not within the scope of this investigation and report, it is pointed out that great care and caution should be exercised in the selection of a portland-pozzolan cement as a sulphate resistant cement. Such cements are not subject to accurate selection by the currently used short-time sulphate tests because such tests are based on functions of expansion which vary widely with the pozzolanic cements. Only after long exposure in strong sulphate solutions produces no corrosion or comparable reduction in strength, may a portland-pozzolan cement be considered dependably resistant. Specifications for duplication of the cement should be specific as to the siliceous blending material and as to the compound composition of the cement clinker. The latter should be as near to that designated in paragraph 6 above as is economically feasible. An admixture should not be depended upon to correct the defects of an unresistant cement.

8. No accelerated tests have been available which can be considered to make the accurate distinctions necessary for a dependable acceptance test.

9. At the present stage of development, the slab-warping test gives promise of dependable selectivity of the relative sulphate resistant ability of unblended portland cements at periods sufficiently short so that, where prolonged tests are not possible, the procedure may be used as an acceptance test with reasonable safety.

10. In selection or specification of a sulphate resistant cement, stipulation of a compound composition which has been proved, for the cement sources under consideration, to impart resistant properties is considered superior to any other method at present available for insuring receipt of a cement having the desired resistance.

11. The above conclusions are further borne out by the results shown in Table 9, of tests of the sodium sulphate resistance obtained with the three different cement specifications under which cement has been purchased by the District. These tests were made from samples of cement manufactured particularly for and actually used in aqueduct work. They clearly show that with proper specifications of chemical compound composition, within practical limits of commercial production, a substantial improvement can be made in the sodium sulphate resistance of the concrete in which it is used. In fact, it is believed that these results give considerable assurance that a dependable sulphate resistant cement can be made.

ACKNOWLEDGMENTS

This work represents a portion of the results of investigations conducted by the Metropolitan Water District of Southern California in connection with construction of the Colorado River Aqueduct. The efforts and services of several persons have contributed so largely to the accomplishment of the work that it is desired particularly to acknowledge their assistance. Credit is due to R. W. Stenzel of the laboratory staff for development of the slab-warping test. Ben E. Nutter was principally responsible for the preparation and care of specimens in the sulphate tests and was generously assisted by other members of the laboratory staff. Prof. Raymond E. Davis of the University of California was consultant and contributed valuable advice and assistance.

Programs of investigation such as this are made at the District's laboratory at Banning and are under the general supervision of Julian Hinds, assistant chief engineer. F. E. Weymouth is general manager and chief engineer of the District, and J. L. Burkholder is assistant general manager.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by February 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for March-April, 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MARCH-APRIL 1937

Discussion of a Paper by Lewis H. Tuthill:

Resistance of Cement to the Corrosive Action of Sodium Sulphate Solutions*

BY ROBERT A. KINZIE, JR.†

It has been a pleasure to study the results of the rather complete series of sodium sulphate tests presented by Mr. Tuthill. The field of sulphate resistant cements has been of particular interest because I have been connected with the development of high silica cements on the Pacific Coast. At the same time the characteristics of modified portland cements made by different manufacturers have been investigated. In the course of this investigation about 1000 specimens of different cements have been tested in sulphate solutions of different concentrations. The methods of test have been as objective as possible, the progress of the reaction usually being followed by noting the change in weight or strength of the specimens tested. This discussion of the work and tests shown by Mr. Tuthill are presented with the view of helping the development of some acceptable measurement of sulphate resistance.

Series SS—The results of the Mr. Tuthill's tests on high silica cements were erratic. The rate of disintegration of any one cement should be either slow or rapid for all of the different methods of curing (Table 1) when compared with another cement under the same conditions of test. The rate of disintegration may vary between the different methods of test.

This is not true in the table. A comparison of cements 9S3 and 10S4 shows that the time of complete disintegration in "San Gabriel-7 Day Cure-in at once wet" was 39 and 29 months; the corresponding time for "Ottawa Sand-28 Day Cure-in at once wet" was 17 and 28

*JOURNAL Amer. Concrete Inst. Nov.-Dec. 1936, p. 83.
†Chemist Santa Cruz Portland Cement Co., Davenport, Calif.

TABLE A—ANALYSES OF HIGH SILICA CEMENT SAMPLES SENT TO METROPOLITAN WATER DISTRICT, LOS ANGELES

Sample Number	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	No. 7	No. 8	No. 9	No. 10	No. 11	No. 12	No. 13	No. 14	No. 15	No. 16
1933 Date of Grinding	Feb. 13–14	Feb. 20–21	Apr. 1–2	Apr. 3–4	Apr. 15–16	Apr. 17–18	Apr. 25–26	May 12–13	June 1–2	July 13–14	July 14–15	July 10–11	July 20–21	July 26–27	Aug. 2–3	Aug. 7–8
Analyses: SiO_2	32.06	32.00	32.60	32.80	31.48	32.10	34.90	31.30	31.40	30.70	32.66	32.64	31.44	31.44	31.40	34.40
Fe_2O_3	2.85	2.42	2.72	2.49	3.18	2.80	2.82	2.72	2.57	2.65	2.57	2.42	2.72	2.57	2.85	2.46
Al_2O_3	7.87	8.28	7.78	8.11	8.06	7.78	5.18	8.20	7.23	8.11	7.53	7.58	7.20	7.52	7.55	5.54
CaO	52.47	53.58	52.28	52.26	51.78	51.58	52.68	52.46	52.98	53.50	52.50	52.20	53.50	53.60	52.60	52.40
MgO			1.84		1.96		1.88	1.88	1.90	2.15	2.27		2.10		2.15	2.45
SO_3	1.29	1.58	1.01	.98	1.69	1.47	1.28	1.26	1.23	1.20	1.16	1.20	1.09	1.13	1.09	1.01
Loss	1.33	1.57	1.92	1.57	1.85	2.05	1.60	1.70	1.85	1.10	1.15	1.35	1.85	1.95	1.75	1.84
Insol.		16.20												14.88		
Fineness: —200 Mesh	97.92	97.04	98.33	98.07	97.95	98.46	97.51	97.17	97.47	97.08	97.58	97.48	97.62	97.54	97.84	97.59
—40 Micron	80.00	74.16	81.36	81.58	81.58	82.86	80.92	79.04	79.08	78.72	79.64	79.76	80.04	79.60	81.20	79.18
—10 Micron			48.24					46.66	45.88	44.98	46.02	47.02	46.76	46.14	49.80	46.40
Set. Time: Initial	2–00	2–10	2–25	1–40	2–20	2–35	1–45	1–25	1–50	2–05	2–05	2–05	2–10	2–30	1–40	1–55
Final	3–45	4–00	4–20	3–45	4–15	5–05	3–45	3–35	4–00	4–00	4–00	4–10	3–53	3–55	3–50	3–50
% Water: Neat	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0
Compound Composition:*																
Tri-Calcium Silicate	56	53	54	58	51	54	59	62	54	56	50	53	52	52	48	54
Di-Calcium Silicate	15	20	18	13	20	18	12	8	18	17	21	20	20	21	24	19
Tri-Calcium Aluminate	16.5	12.9	11	13.7	11.4	12.3	13.7	16.3	14.5	13.9	15.7	14.8	13.8	14.5	15.7	14.2
Tetra Calcium Alumino Ferrite	7.2	7.6	7.6	7.6	8.0	7.2	8.1	7.7	6.0	6.5	6.5	6.7	7.2	7.1	6.8	7.1

*The chemical analyses and physical tests were determined on the samples, which were average samples of a day's grind. The compound composition was calculated fot the clinker which was drawn from the pile for use in grinding the cement. In some cases it was determined on the portland cement that was ground at the same time.

months. While that length of time should be different the ratio between the cements should be fairly constant.

On tests run at this laboratory* on these same cements (condition of test "20-30 Ottawa Sand—28 day cure-in at once wet") the longest time of immersion is 43 months. At present none of the cements shows more than a slight loss on the corners of the specimens. The tests show neither the early disintegration nor the variation between the cements tested. The physical tests, chemical composition and hypothetical compound composition of the clinker used in making the high silica cements are shown in Table A. These results are for cements from 8S2 to 44S18.

The similarity of the cements and the uniformity of their behavior in our tests lead me to believe that some factor of importance was over-looked in the tests reported in Table 1.

Series AD.—Fig. 1 brings out several points of interest. From Table 4 it is seen that the compound composition of the Z clinker (standard) alone would give a cement that would be quickly attacked by sulphate solutions. However, the inclusion of this clinker in a high silica cement gives a product more resistant than either N or H Boulder Dam cements.

The results here considering Z composite, N and H Boulder Dam cements confirm the impression that the results in Table 1 are not representative of the cements tested.

Another point of interest is why the projections of cements S Boulder Dam and S Lab. 2408 beyond the 2-year period are not continuations of the lines between 1 and 2 years. If any change should be made cements R Boulder Dam and S Lab. 2413 should show the increase in slope after two years. However, no data are given to justify this change in slope for any of the cements.

If the data given for the 1 and 2 years—70° F.—10 per cent solution, cements R Boulder Dam and Z Low C_3A clinker, are examined it will be seen that they both show about the same percentage loss in strength between these two periods.

Series X.—This series reproduces more nearly the field conditions encountered in that the solution is weak (1 per cent) and then gradually concentrates as the solvent evaporates. The crystallization of the salt in the concrete duplicates the condition seen in piers and foundations in alkaline soils.

In the last two lines on p. 100 the following sentence occurs, "In other words, no admixture should be depended upon to correct the

*Santa Cruz Portland Cement Co.

defects of an unresistant cement." This cannot be taken too literally, particularly in view of the results of Z composite, H and N Boulder Dam cements in Fig. 1. It is true that for the greatest sulfate resistance the clinker should be inherently resistant.

Slab Warping—This test is a development and improvement of the Merriman test. The Merriman test and the strength results of specimens stored in sodium sulphate give anomalous results. The results shown here indicate that they are more dependable than those of the original Merriman test.

The conclusions present a fair statement of the present knowledge of sulfate resistant cements. Mr. Tuthill is to be commended on the work done.

AUTHOR'S CLOSURE

We are indebted to Mr. Kinzie of the Santa Cruz Portland Cement Co. who identifies the Z brand high silica cement investigated as a product of his company and furnishes the hitherto unavailable hypothetical compound composition of the clinker used in each of the Z brand cements used in the SS series. The following average values may now be placed in the blank spaces in Table 4 for the Z brand composite.

Z BRAND COMPOSITE SAMPLES 1 TO 18, SS SERIES

Hypothetical Compound.....	C_3S	C_2S	C_3A	C_4AF
Maximum.....	62	24	16.5	8.1
Minimum.....	48	8	11.0	6.0
Average.....	54	18	14.1	7.2

Mr. Kinzie states that these cements are of similar compound composition and that their behavior in tests by the Santa Cruz Portland Cement Co. was both uniform and superior. It is his belief that some factor of importance has been overlooked in reporting the tests in Table 1 where he considers the results to be erratic.

It is thought that the term "erratic" is one of opinion as to significant degree commensurate with the type of test. Obviously a disintegration test has a much less determinate end point than such tests as those made for strength or volume change. Perhaps it should have been more definitely stated in the paper that no great effort was made to determine accurately relative *lack* of resistance among cements proved to be unresistant. The general designation of each cement as poor, fair, good, etc. was all that was desired or considered to be of value. Whether a cement disintegrated at 20 or 40 months is relatively unimportant when there are cements that are still intact at those ages.

The similarity of the Santa Cruz cements tabulated by Mr. Kinzie is a matter of viewpoint. From the maximum and minimum values above it would appear that from some viewpoints this group of cements might be considered rather dissimilar.

The superior results with Santa Cruz high silica cement in sulphate resistance in the Santa Cruz laboratory is not questioned but the fact that the mortars made with these cements failed and disintegrated *for some reason* in another series of tests is very impressive when searching for a dependably sulphate resistant cement. It may be that "some factor of importance was overlooked" but it is believed obvious that there is no factor of greater importance than the simple fact that under the described conditions of this test, which were not exceptional, and in the relatively mild 1 per cent solution, mortars made with these cements generally disintegrated while those of cements having compositions low in C_3A generally did not.

In the AD series it is considered a matter of very little importance that the high C_3A clinker Z brand cement showed some superiority to N and H brand Boulder dam cements, inasmuch as all were disintegrated within a year while other compositions, including the low C_3A clinker Z brand, had considerable strength remaining at the end of two years. The fact that the relative time of failure of these unresistant Z, N and H brand cements is not precisely checked in the SS and AD series is not proof that either series is unrepresentative of any of the cements tested to a significant degree because precise measurement of disintegration is not possible or necessary. Again, general classification into poor, fair and good resistance is all that was attempted or is believed to be of real value.

No explanation is offered for the position of dotted extrapolated lines in the figures. They are frankly an estimate. No inference is made or intended that they are otherwise or are substantiated by test values, and the reader is at liberty to rearrange them to his own satisfaction.

It is believed that the results reported are not a condemnation of the Santa Cruz high silica cement, or of any other cement which did not prove to be very resistant to sodium sulphate solutions, because there are many other uses for which these cements would be eminently satisfactory.

Should it be desired to add comparative sulphate resistance to this brand of high silica cement the evidence in the paper shows the way in the results with samples Z 20, 66, 67, 68 and 69 in Tables 6 and 7 and Fig. 2 in which a comparatively resistant clinker was used

having a low percentage of C_3A. It is evident from our tests that this particular pozzolan improves the sulphate resistance of the clinker, but if a sulphate resistant cement is desired there is no point in indicating its improvement of low resistant high C_3A clinker. It is the still superior resistance of the combination with highly resistant low C_3A clinker (see Table 3) that is important if this particular requirement is to be met as adequately as possible with this cement.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . NOVEMBER-DECEMBER 1936

Effect of Calcium and Sodium Chlorides on Concrete When Used for Ice Removal*

(*Progress Report*)

By H. F. Gonnerman,†
MEMBER AMERICAN CONCRETE INSTITUTE

A. G. Timms,‡ T. G. Taylor**

INTRODUCTION

For a number of years rock salt (NaCl) and Calcium Chloride ($CaCl_2$) have been used on grades, at curves, intersections, and around street railway switch points to thaw the ice that forms on pavements during seasons of cold weather. With the increasing use of these two salts, scaling of concrete pavements occurred which prompted the Highway Research Board Committee on Maintenance[1] to make recommendations for treating icy pavements with cinders and sand impregnated with salt solutions to avoid heavy concentration of the salts. These recommendations have not been generally observed and following the abnormally severe winter of 1935-36, when more frequent applications were made and probably much larger quantities of the chlorides than usual were used for melting ice, serious scaling was observed on many concrete pavements. Maintenance engineers have reported that at switch points on street railway lines, the salt covering on the pavement was at times as much as ¼-in. thick. The studies reported here were prompted by the circumstances just described.

SCOPE OF TESTS

These studies have been divided into two parts. The first deals with methods of treating the surface of existing concrete pavements

*Received by the Institute Secretary, Nov. 10, 1936.
†Manager of Laboratory, Portland Cement Association.
‡Assistant Manager of Laboratory, Portland Cement Association.
**Assistant Engineer, Portland Cement Association.
[1]See page 336, Part I, Proceedings Highway Research Board, 1933.

to prevent scaling, and the second deals with the factors which reduce the tendency of concrete surfaces to scale when subjected to calcium and sodium chloride applications. This progress report deals primarily with methods of treating concrete surfaces to protect them against the action of the chlorides.

Preliminary Tests. A 12 by 12 by 2½-in. concrete slab provided with a dyke around the top surface was selected as the test specimen.[2] In selecting a test cycle for the investigation, preliminary tests were carried out on 6 slabs made of a paving mixture containing 6 gal. of water per sack of cement and having a floated finish similar to that used on pavement surfaces. Two hours after placing the concrete, dykes of 1:2 mortar by weight were constructed around the edges of the slabs to hold the water or solutions placed on the finished surfaces.

In these preliminary tests three procedures were tried as follows:

(1) Fresh water (250 ml.) was frozen on the surface of the slab during 18 hours in a refrigerator at a minimum temperature of —18° F. Upon removal from the refrigerator, equal volumes of commercial calcium chloride or of sodium chloride were placed directly on the ice. The quantities used were 85 grams of calcium chloride and 115 grams of rock salt which correspond to 2.4 and 3.2 lb. per sq. yd. respectively. These quantities do not exceed those frequently used in practice. The thawing period was continued for 6 hours. During the last hour of the thawing period, the solution was drained off, the surface brushed with a wire brush, and fresh water (250 ml.) again applied. The slab was then replaced in the refrigerator for 18 hours. This cycle was repeated every day in the week except over the week-end, when the specimen was left in the refrigerator for 42 hours.

(2) A 12 per cent solution of either sodium or calcium chloride was frozen on the surface of the concrete during 18 hours in the refrigerator at a minimum temperature of —18° F. The specimen was then allowed to thaw in the air of the laboratory for 6 hours. During the last hour the solution was drained off and a full strength solution replaced on the surface before the next freezing.

(3) A slab having a wet surface but no excess water, was frozen for 18 hours and the frozen surface then thawed with a 12 per cent sodium or calcium chloride solution while exposed in the air of the laboratory. After 5 hours the solution was drained off and fresh water allowed to stand on the slab for 1 hour. The excess water was then drained off and the specimen again frozen.

After carrying these preliminary tests through 25 or 26 cycles of freezing and thawing, it was decided to use procedure (1) in which the dry commercial salts were applied directly to ice previously frozen on the concrete. This cycle was chosen because it approximates the cycle which most frequently occurs in the field and, with the concentrations of chemical used, it showed the greatest amount of surface attack for both calcium and sodium chloride and was therefore, in the nature of an accelerated test. The concrete slabs for the preliminary tests just described, were cured in the moist room for 7 days and

[2]See Report of Committee on Maintenance, Treatment of Icy Pavements, p. 333, Proceedings Highway Research Board, Part I, 1933.

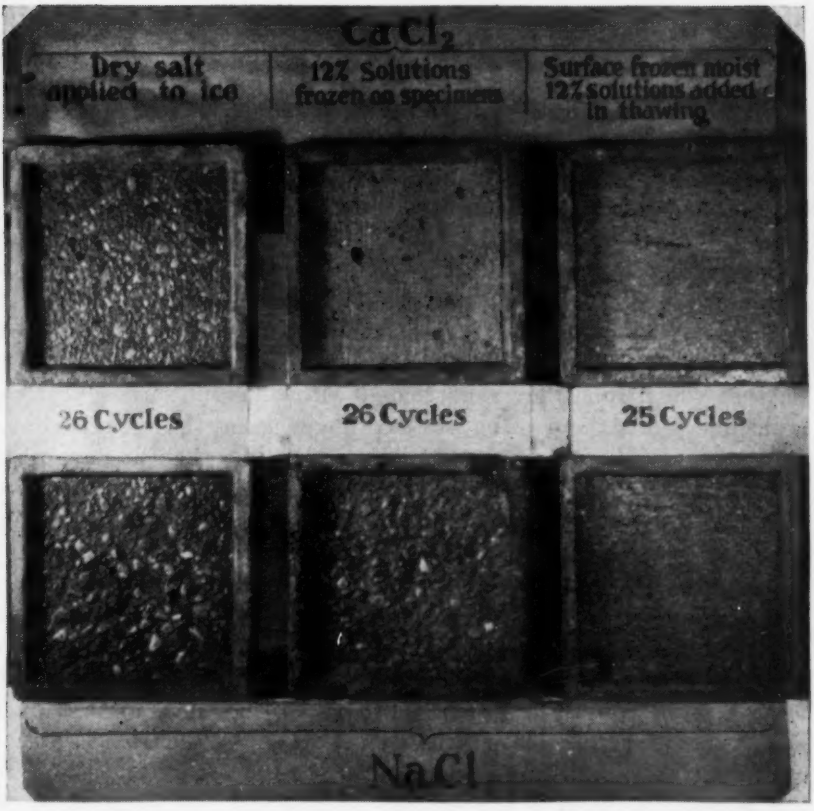


FIG. 1—SPECIMENS IN PRELIMINARY INVESTIGATION TO DETERMINE TEST PROCEDURE

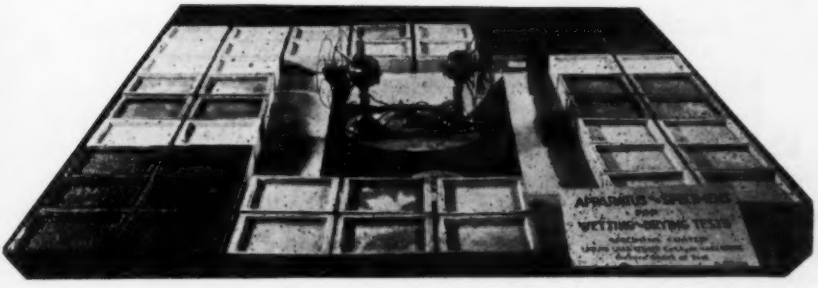


FIG. 2—WETTING AND DRYING TESTS

then in air of laboratory for 7 days before placing in the refrigerator. A photograph of these slabs at the end of 26 cycles is reproduced in Fig. 1.

Materials and Methods for Main Investigation. Aggregates for the concrete consisted of sand and gravel from Elgin, Illinois, graded up to 1½ in. For the slabs subjected to alternate freezing and thawing, chert and other non-durable particles were, so far as possible, removed from the coarse aggregate by hand. The cement was the laboratory mixture consisting of equal parts of four brands of portland cement purchased in Chicago. The mix in all cases was 1:3.16:4.96 by weight.

In the main investigation the slabs were moist-cured for 7 days. They were then allowed to remain in the air of the laboratory for 21 days at which time they were coated with different surface coatings and subjected to the freezing and thawing cycles at the age of 35 days. In a few cases the specimens were somewhat older than 28 days when coated.

In addition to the freezing and thawing cycles, duplicate specimens for many of the surface coatings were subjected to an alternate wetting and drying test. In this test 12 per cent solutions of the chlorides were applied to the surfaces of the specimens and a current of air from electric fans allowed to blow over them (Fig. 2). The fans were mounted on a separate revolving table so that the air over the specimens was in constant motion. The specimens were rotated once weekly and their relative positions on the table also changed so that each received as nearly uniform treatment as possible over the entire test period. As soon as the surfaces became dry, the solutions were replenished. After 5 weeks one of the companion specimens for each type of coating was placed in a pan of water to a depth of 1½ inches and wetting and drying of the top surface continued.

Both the commercial calcium chloride and rock salt used were purchased from local sources. Before use, each lot was analyzed to see

TABLE 1—CHEMICAL ANALYSES OF CALCIUM AND SODIUM CHLORIDES
Each value is the average of two determinations.

Lot No.	Material	Per Cent by Weight								Calculated Composition			
		SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO	SO_3	Na_2O	Cl	$CaCl_2$	NaCl	$CaSO_4$	H_2O+ Undet.
13360	NaCl	0.04	Tr.	Tr.	0.75	Tr.	0.98	.....	59.56	Tr.	98.2	1.7	.....
13448	NaCl	Nil	Tr.	Tr.	0.50	Tr.	0.68	.....	60.00	Tr.	98.9	1.2	.....
13490	NaCl	Tr.	Tr.	Tr.	0.30	Tr.	0.49	.....	59.96	Tr.	98.8	0.8	0.4
13359	$CaCl_2$	Tr.	Tr.	Tr.	39.25	Tr.	Tr.	0.85	50.14	77.0	1.6	Nil	21.4
13489	$CaCl_2$	Nil	Tr.	Tr.	39.35	Tr.	Tr.	0.90	50.76	77.8	1.7	Nil	20.5

whether it contained any foreign material.* The chemical analyses (Table 1), showed only a trace of sulfate in the calcium chloride and about 1 per cent in the sodium chloride.

RESULTS OF TESTS

Surface Coatings. Table 2 gives the surface coatings tried, the cycle at which scaling started, and the rating of the surface of the different specimens after being subjected to 10, 25 and 50 cycles of freezing and thawing. Table 3 lists the specimens and their surface condition after 8 and 14 weeks exposure to wetting and drying.

The best record of the condition of the surface of the slabs is in photographs taken after different cycles of freezing and thawing or after a given exposure to alternating wetting and drying. Photographs covering typical test conditions only are reproduced in this paper. Fig. 3 shows typical surfaces before test and after bad scaling had occurred. In Fig. 4 to 6 are photographs of several slabs after various cycles of freezing and thawing.

The relative worth of the different surface coatings can be ascertained from Tables 2 and 3. Of the many materials tried, only two, boiled linseed oil and soybean oil can be considered of value in protecting the concrete against scaling. Whereas with most of the coatings tried scaling occurred between 3 and 5 cycles, with linseed oil it did not occur until 70 cycles with the sodium chloride, and in the case of the calcium chloride it has not occurred to date (100 cycles). Slabs treated with soybean oil have withstood over 40 cycles to date without scaling. A few of the other coatings notably Hunt Process (clear), Pyroil, stearic acid, and waste crankcase oil delayed the start of scaling somewhat but in most cases did not protect against serious scaling much beyond 25 cycles.

The linseed oil coating, which thus far has been superior to all the others, consisted of a mixture of 50 per cent boiled linseed oil and 50 per cent turpentine applied to the surface of the slab with a brush, followed 24 hours later with a coat of undiluted linseed oil. Boiled linseed oil was used because the oil is boiled in the presence of a lead salt which causes the oil to dry and oxidize better than raw oil. In drying and oxidizing, the oil appears to form a seal below the surface of the concrete which prevents the entrance of water or salt solutions. Slabs coated with linseed oil in the manner described showed no evidence of attack by the chlorides until the 70th cycle, at which

*Salt used for thawing purposes has occasionally been found to contain a high percentage of sodium sulfate. In one instance a waste product salt from a chemical process was donated to a city for use in melting ice. Chemical analysis showed the salf to contain 15 per cent sodium sulfate and where it was used, very severe scaling of a concrete pavement resulted.

TABLE 2—FREEZING AND THAWING TESTS

Tests made by Procedure No. 1—clear water frozen on surface, thawing accomplished by application of dry sodium or calcium chloride.

In general two slabs made for each condition of test. Values represent tests on individual specimens.

Values in parentheses give number of cycles when test discontinued because specimen very badly scaled.

Extent of scaling of surface of slabs indicated by following ratings:

0—No scale
1—Slight scaling
2—Slight to moderate scaling
3—Moderate scaling
4—Moderate to bad scaling
5—Bad scaling

Ref. No.	Surface Treatment	Cycle at Which Scaling Started		Rating of Condition of Surface after Cycles of Freezing and Thawing Indicated							
				NaCl applied to Ice				$CaCl_2$ applied to Ice			
		NaCl	$CaCl_2$	10	25	50	75	10	25	50	75
	Slabs 1 day in molds, 6 days in moist room, 21 days in air. At age of 28 days slabs coated, then stored 7 days in air before being subjected to freezing and thawing.										
[100	None	3rd		1	3	4	No chlorides applied to surfaces of these two specimens]				
[111	None	3rd		1	3	4					
49–50	None	2nd	2nd	2	5	(32)*	–	2	5	5	(50)
73–74	None	2nd	2nd	1	5	(32)	–	1	3	(32)	–
55–56	Boiled linseed oil; 1st coat equal parts oil and turpentine; 2nd coat 100% oil	71st	–	0	0	0	1	0	0	0	0
79–80		70th	–	0	0	0	1	0	0	0	0
162–163	Boiled linseed oil; 1st coat equal parts oil and benzine; 2nd coat 100% oil			0	0	**		0	0	**	
164–165	Boiled linseed oil; 1st coat equal parts oil and gasoline; 2nd coat 100% oil			0	0	**		0	0	**	
166–167	3 parts linseed oil; 1 part turpentine. 1 coat			0	0	**		0	0	**	
168–169	Boiled linseed oil. 1 coat			0	0	**		0	0	**	
150–151	Refined Soybean oil. 1st coat equal parts oil and Japan dryer; 2nd coat 100% oil			0	0	**		0	0	**	
154–155	Refined Soybean oil; 1st coat equal parts oil and turpentine; 2nd coat 100% oil			0	0	**		0	0	**	
152–153	Pyroil	14th	20th	0	1			0	1		
156–157	Hunt process (clear) 1 coat	9th	14th	1	2	(35)	–	0	2		
57–58	Waste crankcase oil. 2 coats	4th	6th	1	3	4	(63)	1	2	3	4
81–82		4th	3rd	1	1	2	(50*)	1	1	2	2
59–60	1 part stearic acid, 9 parts carbon tetrachloride. 2 coats	6th	6th	1	1	5	(51)	1	1	5	(51)
83–84		10th	10th	1	1	(43)	–	1	1	5	(50)
63–64	Emulsified asphalt. 1 coat	5th	5th	1	5	(25)	–	1	3		(25)
87–88		7th	7th	1	2	5	(50)	1	1	5	(50)
65–66	1 part asphalt, 1 part gasoline. 1 coat	5th	3rd	1	5	(25)	–	1	3	(25)	
89–90		4th	4th	1	2	5	(50)	1	2	5	(50)
67–68	1 part paraffin, 9 parts naphtha. 4 coats	5th	5th	1	4	(41)	–	1	2	(32)	–
91–92		3rd	9th	1	3	(33)	–	1	2	(33)	–
69–70	Hot paraffin. 1 coat	3rd	3rd	2	5	(25)	–	2	4	(25)	–
93–94		3rd	3rd	1	5	(25)	–	1	4	(25)	–
61–62	4 parts Tarmac, 1 part naphtha 1 coat	3rd	3rd	1	4	(25)	–	1	3	(25)	–
85–86		3rd	3rd	1	2	5	(50)	1	2	5	(50)

*Discontinued due to failure of dyke and very bad scaling.

**Rating still 0 at latest inspection (40 cycles).

TABLE 2—*Continued*

Ref. No.	Surface Treatment	Cycle at Which Scaling Started		Rating of Condition of Surface after Cycles of Freezing and Thawing Indicated							
				NaCl applied to Ice				$CaCl_2$ applied to Ice			
		NaCl	$CaCl_2$	10	25	50	75	10	25	50	75
71–72	Sodium silicate, 1 coat	3rd	3rd	1	4	(32)	–	1	2	4	(64)
95–96		5th	3rd	1	3	(38)	–	1	3	(38)	–
170–171	Colloidal sulfur in water. 1 coat	3rd	3rd	3	5	(29)	–	3	3		
11AA–12AA	Light spraying oil. 2 coats	3rd	3rd	2	5	(27)	–	2	5	(27)	–
51–52	Calcium bicarbonate solution, 6 applications	2nd	2nd	1	4	(25)	–	1	4	(25)	–
75–76		2nd	2nd	1	5	(25)	–	1	4	(25)	–
53–54	Carbon dioxide gas in water, 6 applications	2nd	2nd	1	4	(25)	–	1	4	(25)	–
77–78		3rd	2nd	1	5	(25)	–	1	5	(25)	–
	Slabs 1 day in molds, 6 days in moist room, 21 days in air. At age of 28 days slabs coated and then partly buried in moist sand for 6 days before being subjected to freezing and thawing, with bottom placed in 1½-in. water.										
101–102	None	2nd	2nd	4	5	(37)	–	4	5	(37)	–
103–104	Boiled linseed oil. 1st coat equal parts oil and turpentine; 2nd coat 100% oil	–	51st	0	0	0		0	0	0	
105–106	1 part stearic acid, 9 parts carbon tetrachloride. 2 coats	6th	6th	2	4	(37)	–	2	4	(37)	–
107–108	Emulsified asphalt	3rd	3rd	2	3	(37)	–	2	4	(37)	–
109–110	1 part Benite, 5 parts naphtha. 2 coats.	9th	9th	1	2	(47)	–	1	1	(47)	–
	Slabs 1 day in molds, 6 days in moist room, 18 days in air, 3 days immersed in water. At age of 28 days surfaces of slabs dried 4 hrs., then coated and slabs partly buried in moist sand 6 days before being frozen and thawed, with bottoms placed in 1½-in water.										
112	None	4th		1	1	4	No chlorides applied to surfaces of these two specimens				
123		3rd		1	2	(45)*					
113–114	None	4th	4th	4	5	(34)	–	3	5	(34)	–
115–116	Boiled linseed oil. 1st coat equal parts oil and turpentine. 2nd coat 100% oil	19th	23rd	0	2	(43)	–	0	1	(43)	–
117–118	1 part stearic acid, 9 parts carbon tetra chloride. 2 coats	7th	7th	2	5	(34)	–	2	4	(34)	–
119–120	Emulsified asphalt	3rd	3rd	2	5	(34)	–	1	4	(34)	–
121–122	1 part Benite, 5 parts naphtha 2 coats.	7th	12th	1	2	(33)	–	0	1	(43)	–

*Discontinued due to failure of dyke and very bad scaling.

time one of the slabs thawed with sodium chloride showed scaling which started as a pop-out over one of the larger aggregate particles (Fig. 6). The companion slab first scaled at the 71st cycle. As the freezing and thawing of these two slabs continues the scaling appears to be progressive. On the other hand the two corresponding slabs, thawed with calcium chloride, have shown no scaling to 100 cycles.

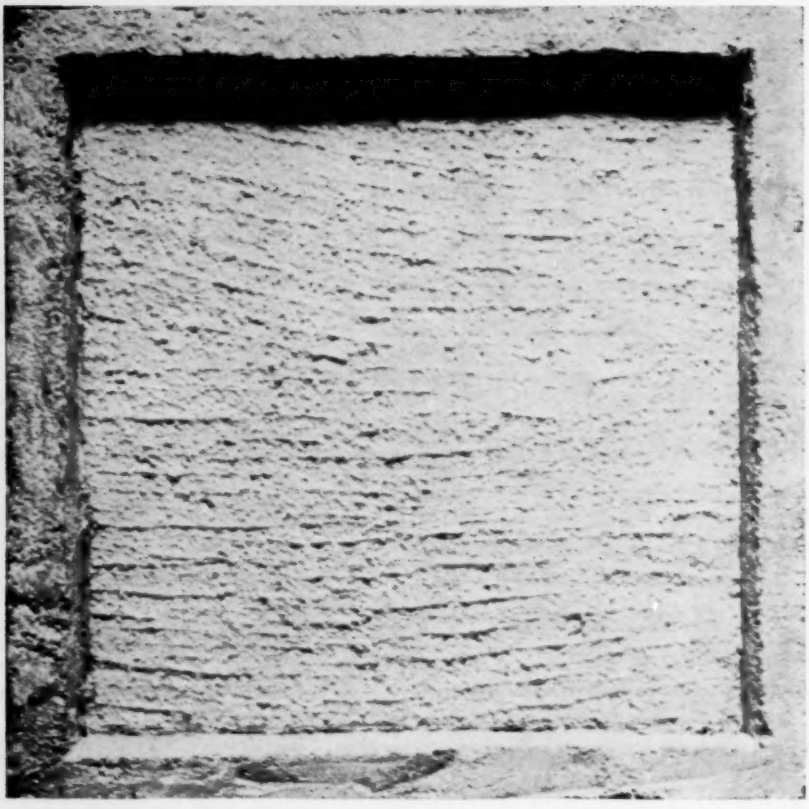


FIG. 3 (A)—TYPICAL SURFACE BEFORE TEST (SEE OPPOSITE PAGE)

The favorable showing of the linseed oil raised a number of questions, such as obtaining a cheaper diluent in place of turpentine. Slabs were coated with linseed oil using gasoline or benzine in place of turpentine. A few tests were also made to determine the amount of turpentine needed in the priming coat in order to get sufficient penetration to withstand the action of the chlorides. All of these latter specimens have been subjected to 25 cycles of freezing and thawing and so far show no evidence of scaling.

To determine whether the wear caused by automobile chains during icy weather might impair the effectiveness of the linseed oil treatment, two slabs coated with linseed oil were frozen and thawed for 10 cycles and the surfaces then ground with an emery wheel until the brush marks had been removed. These slabs without further treatment with linseed oil, were subjected to an additional 15 cycles without showing

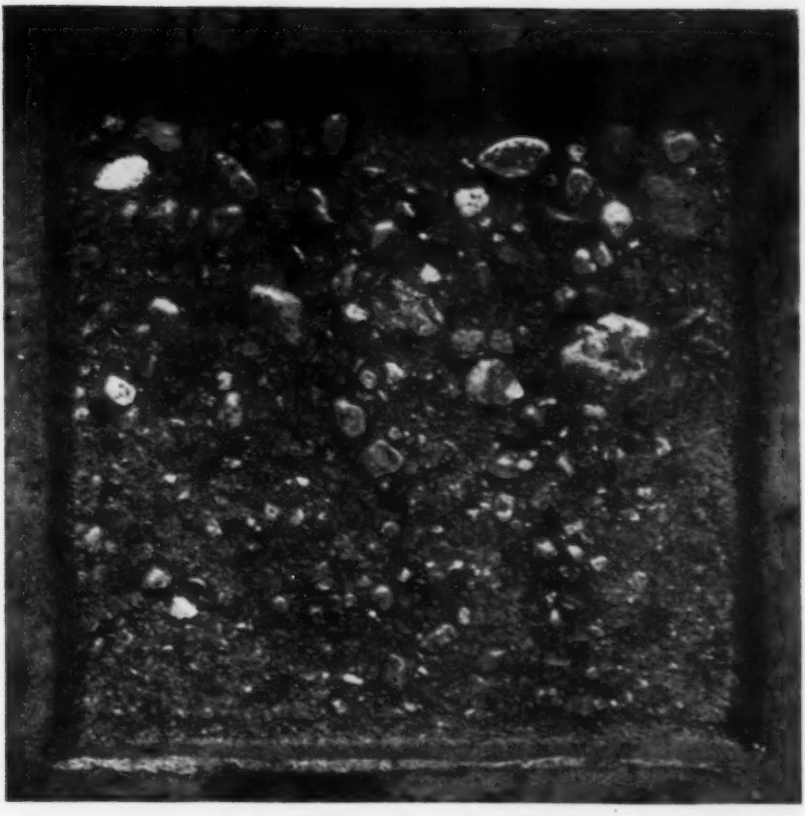


FIG. 3 (B)—TYPICAL SURFACE AFTER 50 CYCLES OF FREEZING AND THAWING (SEE OPPOSITE PAGE)

any scaling, and are now undergoing further cycles of freezing and thawing.

Since linseed oil is rather expensive, tests were made with soybean oil which has properties similar to linseed oil, but is about 25 per cent cheaper. Several forms of soybean oil were obtained, unrefined, refined, blown, and oil sold as clear cement paint vehicle. These oils have been used with and without turpentine on slabs which have received over forty cycles without showing any evidence of scaling.

Tests of Slabs Frozen with Bottoms Wet. Specimens cured and coated in the usual manner were placed in moist sand for 6 days. Then the bottoms of the slabs were placed in about 1½ in. of water and the top surfaces frozen and thawed as before. Except in the case of the linseed oil, these specimens scaled sooner than corresponding speci-

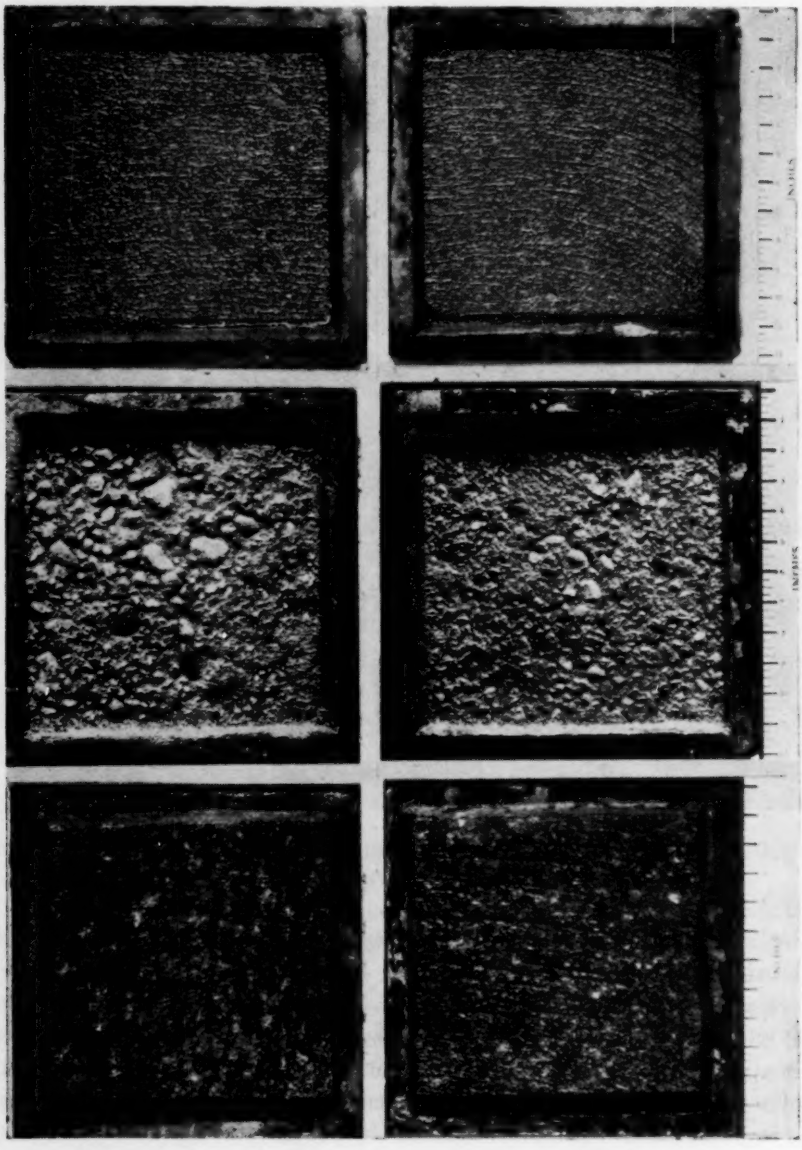


FIG. 4—TYPICAL SURFACES OF SLABS WITH DIFFERENT SURFACE COATINGS AFTER 25 CYCLES OF FREEZING AND THAWING

Rock salt applied to ice on specimens at left—flake calcium chloride applied to ice on specimens at right. Coatings: Top pair of specimens; soybean oil (turpentine); center pair, no coating; bottom pair, cut back asphalt.

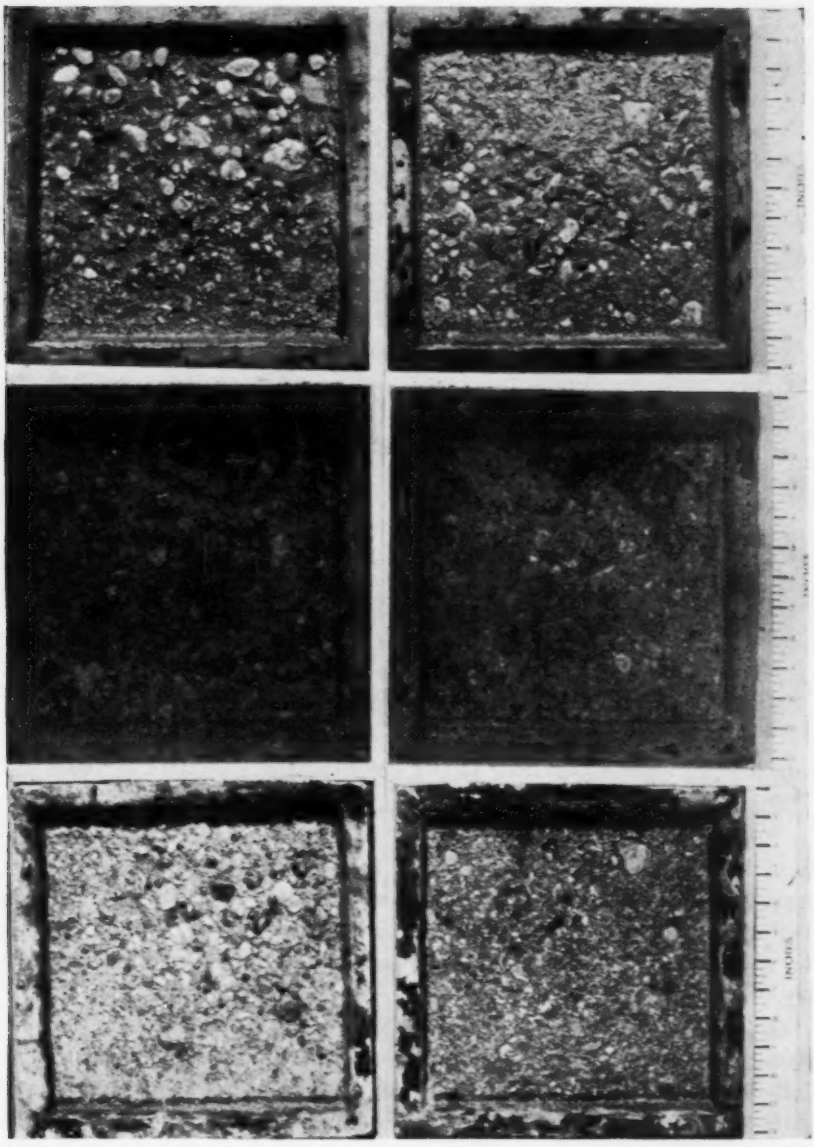


FIG. 5—TYPICAL SURFACES OF SLABS WITH DIFFERENT SURFACE COATINGS AFTER 50 CYCLES OF FREEZING AND THAWING

Rock salt was applied to ice on three specimens at left—flake calcium chloride to ice on specimens at right. Surface coatings were: Top pair of specimens, stearic acid; center pair, Emulsified asphalt; bottom pair, sodium silicate.

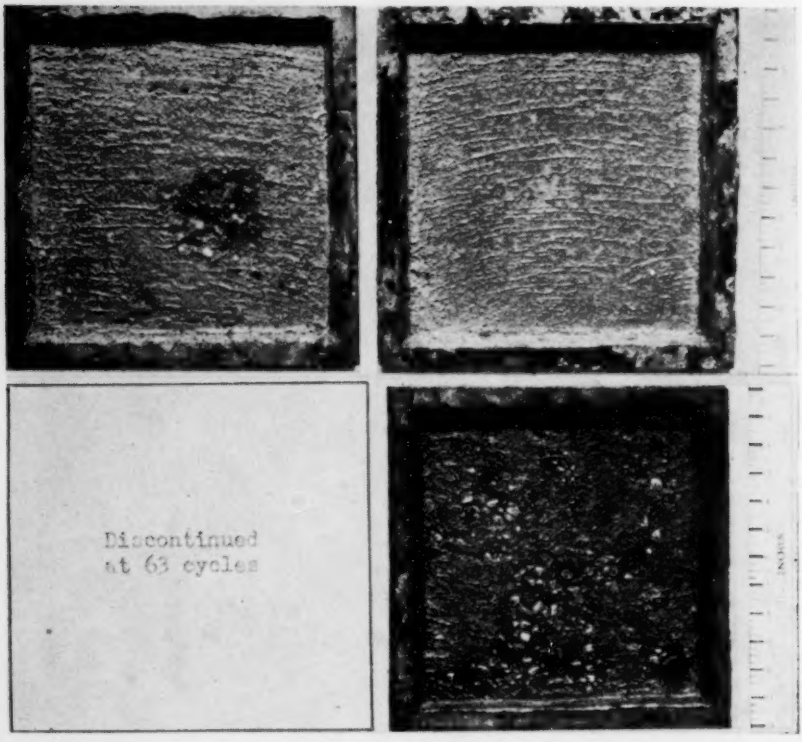


FIG. 6—SURFACE CONDITION AFTER 75 CYCLES FREEZING AND THAWING

Rock salt applied to ice on specimens at left; flake calcium chloride to ice on specimens at right. Top pair of specimens treated with linseed oil; bottom pair, with old crank case oil.

mens frozen and thawed out of water in the usual manner. Specimens coated when the surfaces were wet and then frozen with the bottoms resting in water were more affected by the application of the chlorides than specimens tested in a similar manner but coated when thoroughly dry.

Tests of Old Concrete. Pieces of concrete taken from a pavement about six years old were given the same treatment as the small test slabs in order to check the general observation that concrete several years old is not readily affected by the application of sodium or calcium chloride. That this observation was substantiated, at least so far as these specimens is concerned, is shown by the excellent condition of this old concrete after 50 cycles (Fig. 8).

It was thought that the better resistance of the old concrete to salt attack might be due either to sealing of the surface by oil drippings, or to carbonation of the surface which reduced absorption and pre-

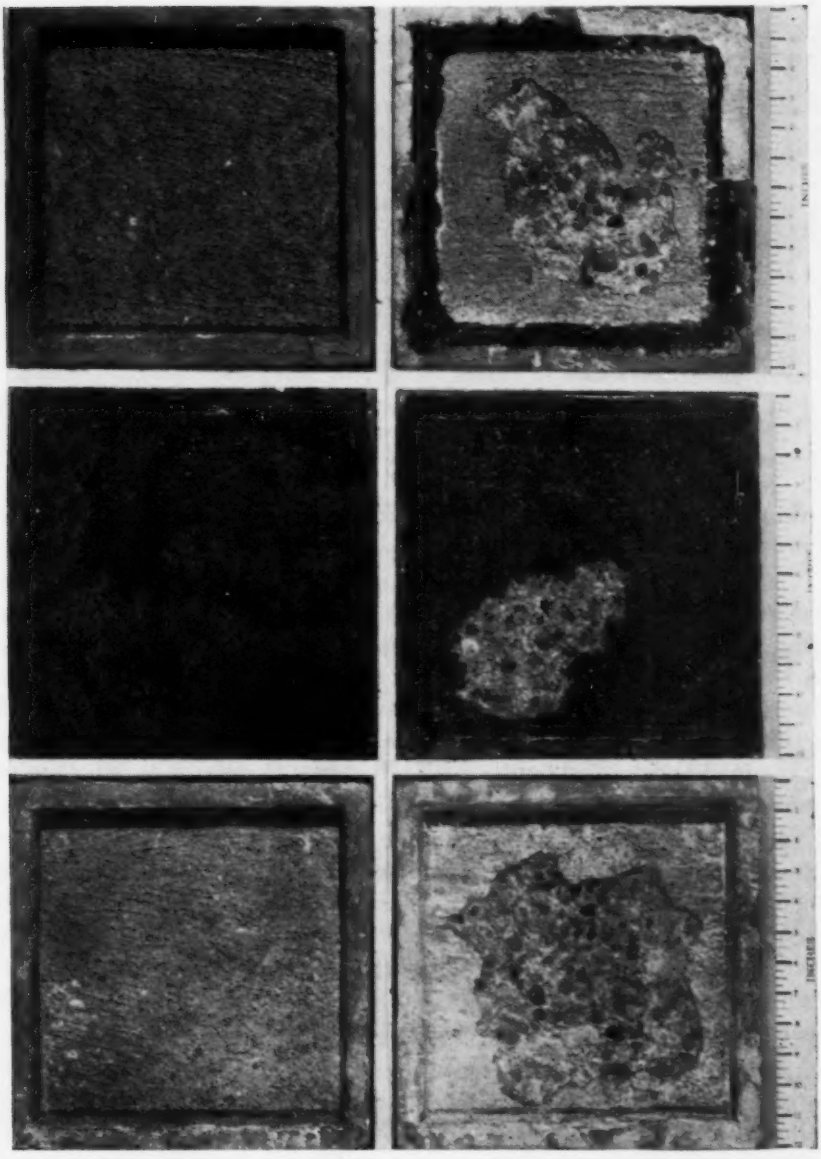


FIG. 7—TYPICAL SCALE DEVELOPED ON SLABS EXPOSED TO ALTERNATE WETTING AND DRYING FOR EIGHT WEEKS

12 per cent NaCl solution applied to surface of three specimens at left; 12 per cent $CaCl_2$ solution to specimens at right. Top pair of specimens had no protective coating; center pair, cut back Tarmac; bottom pair, sodium silicate.

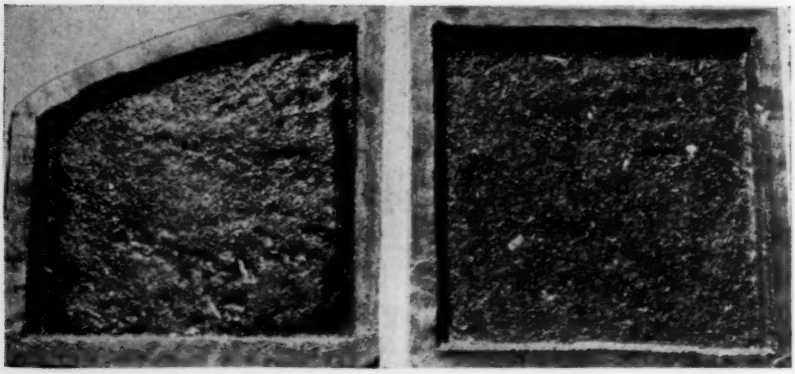


FIG. 8—SECTIONS OF CONCRETE PAVEMENT 6 YEARS OLD AFTER 50 CYCLES OF FREEZING AND THAWING BY PROCEDURE 1

(Left) Rock salt applied to ice. (Right) Flake calcium chloride applied to ice.

vented penetration and attack of the salts. An attempt was made to produce carbonation but the methods tried did not delay scaling.

Wetting and Drying Tests. Specimens made, cured, and coated in the usual manner were given repeated applications of 12 per cent solutions of sodium or calcium chloride, each application being followed by drying in an air current from an electric fan (Table 3). The slabs were exposed for 7 weeks to this treatment. They were then scrubbed with fresh water, allowed to dry for one week, and then carefully examined and tapped lightly with a hammer to determine whether or not the top surface had separated from the adjacent concrete.

Fig. 7 shows photographs of some of these slabs eight weeks after the start of the tests. It will be noted that only the calcium chloride solution produced bad scaling. When examined, the top mortar layer of many of the slabs was found to have loosened so that it could readily be removed in the form of a relatively thick (⅛ to ¼-in.) scale. Approximately one-third of the total number of slabs treated with calcium chloride solution showed this type of scaling.

The specimens treated with the 12 per cent sodium chloride solution when examined 14 weeks after the start of the tests showed a very thin scale and exposed sand particles. From these wetting and drying tests it appears that common salt is less harmful than calcium chloride when freezing does not occur.

CONCLUDING REMARKS

The tests described in this progress report show that relatively young concrete (35 days old), of normal paving mix and without a

TABLE 3—WETTING AND DRYING TESTS

Slabs 1 day in molds, 6 days in moist room, 21 days in air. At age of 28 days surface of slabs treated as indicated, then stored 7 days in air of laboratory before being subjected to repeated applications of 12% solutions of sodium or calcium chloride, each application being followed by drying in blast of air from electric fan.

In general two slabs made for each condition of test. Values represent tests of individual specimens

Ref. No.	Surface Treatment	Scaling of Surface after Exposure Indicated			
		8 Weeks		14 Weeks	
		NeCl	$CaCl_2$	NaCl	$CaCl_2$
1–2	None	Slight	Bad	Slight	–
25–26*		Slight	Bad	Slight	–
3–4	Calcium bicarbonate solution, 6 applications	Slight	Bad	Slight	–
27–28*		Slight	Bad	Slight	–
5–6	Carbon dioxide gas in water, 6 applications	Slight	Bad	Slight	–
29–30*		Slight	Bad	Slight	–
7–8	Boiled linseed oil. 1st coat equal parts oil and turpentine.	None	None	None	None
31–32*	2nd coat 100% oil	None	None	None	None
9–10	Waste crankcase oil. 2 coats	Slight	None	Slight	None
33–34*		Slight	None	Slight	None
11–12	1 part stearic acid, 9 parts carbon tetra chloride. 2 coats	Slight	None	Slight	None
35–36*		None	None	Slight	None
13–14	4 parts Tarmac, 1 part naphtha. 1 coat	None	Bad	None	–
37–38*		None	Bad	None	–
15–16	Emulsified asphalt. 1 coat	Slight	None	Slight	None
39–40*		None	None	None	None
17–18	1 part asphalt, 1 part gasoline. 1 coat	Slight	None	Slight	Bad
41–42*		None	None	None	None
19–20	1 part paraffin, 9 parts naphtha. 4 coats	Slight	Bad	Slight	–
43–44*		Slight	Bad	Slight	–
21–22	Hot paraffin. 1 coat	Slight	Slight	Slight	Slight
45–46*		Slight	Slight	Slight	Bad
23–24	Sodium silicate	Slight	Slight	Slight	None
47–48*		Slight	Slight	Slight	None

*After 5 weeks of the wetting and drying treatment bottoms of these specimens placed in water to a depth of 1½ in. and the wetting and drying of the top surface continued.

protective coating, will scale badly due to repeated removal of ice with either sodium chloride or calcium chloride. Likewise, the repeated application of solutions of these salts to the bare surface with subsequent drying and without freezing, results in surface scaling.

Where the use of sodium chloride or calcium chloride is considered necessary, either for ice removal or in combination with granular materials to provide traction, it is recommended that the pavement be thoroughly brushed and then treated with boiled linseed oil thinned by an equal volume of turpentine, at the rate of about 50 sq. yd. per gallon, and that this be followed in 24 hours by an application of neat boiled linseed oil at the rate of about 70 sq. yd. per gallon. The pavement should be reasonably dry when the applications of oil are made.

Tests of other surface coatings and methods of application are under way. Tests are also being made in the second phase of the investigation which deals with factors, other than surface coatings, that reduce the tendency of concrete surfaces to scale when subjected to the application of calcium or sodium chloride. The latter involves a study of methods of improving the resistance to scaling by changes in the characteristics of the concrete itself.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by February 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for March-April, 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . NOVEMBER-DECEMBER 1936

Report of Committee 313

Effect of Plastic Flow and Volume Changes on Design*

CLYDE T. MORRIS, CHAIRMAN†

THE routine designer in an architect's or an engineer's office is usually, and necessarily so, a follower of accepted methods and standard specifications. These methods of calculation and specifications are the outgrowth of experience and research and are being constantly added to and altered as knowledge of the properties of the materials increases. Anyone who has had experience in testing knows that the actual stress distribution in a simple reinforced concrete beam does not agree with the accepted theory, but it is also well known that this usual theory, when used with properly selected unit stresses gives results that are safe in spite of the fact that the actual unit stresses may be entirely different.

One of the duties of this Institute is to take the products of research and work them over into simple methods and specifications which can be used by the routine designer in producing a safer, a more economical or a more enduring structure.

The work of those two outstanding committees of the Institute, the one on plastic flow under the direction of Prof. Raymond E. Davis,(11), (12), (13), (14), (26)‡ and the one on columns with Professors F. E. Richart and Inge Lyse in charge,(18), (19) has given us a wealth of data and a new insight into the behavior of concrete and reinforced concrete under sustained stress. It now behooves us to take these data and see what changes in design procedure and specifications may be necessary to bring these abreast with our present knowledge of the materials.

*Received by the Institute Secretary, Nov. 1, 1936.

†Professor of Civil Engineering, Ohio State University. Other members of Committee: F. R. MCMILLAN, J. R. SHANK, CHARLES S. WHITNEY.

‡Numbers in parentheses refer to bibliography at end of paper by Prof. J. R. Shank, "The Mechanics of Plastic Flow of Concrete," JOURNAL, American Concrete Institute, Nov.-Dec. 1935; *Proceedings* Vol. 32, p. 178.

It has long been recognized that concrete changes in volume when subjected to *stress* (elastic deformation), *sustained load* (plastic flow) *temperature changes*, and *changes in moisture content* (shrinkage), and many investigators have worked upon the determination of the magnitude of these volume changes. The literature upon the subject is voluminous, but little so far has been published upon the utilization of these data in design. Prof. J. R. Shank, in his paper "The Mechanics of Plastic Flow of Concrete"* has summarized the work so far published on plastic flow, and the bibliography at the end of his paper is comprehensive. In what follows frequent reference will be made to Professor Shank's paper.

In the theoretical design of structures it is usually assumed that the effects of changes in volume upon the stresses is limited or controlled by the introduction of expansion joints at certain points.

Errors in the location of these expansion jonts and in their design have caused numerous unsightly cracks in otherwise well proportioned structures and the gradual disintegration of the concrete due to these cracks.

Concrete is a material which lends itself to monolithic construction and the presence of expansion joints is often a detriment to the usefulness and economy of the structure. In such cases volume changes must be prevented or provided for by proper design and reinforcing. It is the purpose of this paper to indicate how the volume changes in certain simple members and structures will affect the stress distribution in them, and the amount and location of the reinforcing. For data as to the magnitude of volume changes due to various causes the reader is referred to the bibliography at the end of Professor Shank's paper already cited.

1—SHRINKAGE

Observations tend to support the conclusion that shrinkage of concrete during curing is caused primarily by loss of water, or drying out. This shrinkage is necessarily a function of time, relative area of exposed surface to volume, humidity, etc. In estimating the amount of shrinkage which will occur it is necessary to take all of these factors into consideration. But when a design is being made it is usually difficult to predict the moisture conditions and exposure which the structure will have to withstand.

Usually in bridge superstructures a condition of equilibrium will have been reached within three or four months after placing, although this may be delayed by wet weather. In buildings the greater part of the shrinkage will take place during the first heating season. Modern

*JOURNAL, Am. Concrete Inst., Nov.-Dec. 1935, *Proceedings* Vol. 32, p. 149.

air conditioning systems which humidify the air in winter should tend to reduce this action.

The initial shrinkage (first 60 to 90 days) may be as much as .001 when the relative humidity is very low as in the interiors of heated buildings. For structures exposed to the weather the relative humidity will rarely be below 70 per cent for any length of time and the initial shrinkage will not exceed .0004 to .0006 except in arid desert regions. For hydraulic structures or foundations which are entirely immersed in water or moist earth the shrinkage will be negligible.

2—PLASTIC FLOW

Prof. J. R. Shank has summarized the present day knowledge of plastic flow as follows:

(a) In general the plastic flow of concrete may be expressed by an equation of this form,

$$y = C\sqrt[a]{x} \quad \ldots\ldots (1)$$

in which y = the plastic flow in millionths of an inch per inch for a unit stress of one pound per sq. in.

x = time after loading in days.

a and C = constants varying with the material.

(b) The shape of the curve as expressed by equation (1) is fairly accurate under controlled conditions for times up to one year after loading; for longer periods y should be progressively reduced until, after a period of five years no further plastic flow need be considered. The rate of plastic flow in air is quite sensitive to changes in humidity.

(c) The plastic flow of concretes loaded at 28 days for periods of five years or more will not exceed the flow at one year by more than 30 per cent, and for concretes loaded at 3 months the excess over the flow at one year may reach 40 per cent.

(d) Plastic flow data so far available may be expected to be as much as 50 per cent in error when considering external conditions and any specific concrete mixture.

(e) The plastic flow for ordinary portland cement concretes made from ordinary aggregate materials, natural sands, natural gravels and limestones, loaded in air at 28 days may be expressed with sufficient accuracy by

$$y = 0.13\sqrt[3]{x} \quad \ldots\ldots (2)$$

(f) For the same concretes in water equation (1) may be written

$$y = .089\sqrt[4]{x} \quad \ldots\ldots (3)$$

For other materials and conditions the reader is referred to Professor Shank's paper previously cited.

3—REINFORCED CONCRETE MEMBERS

Shrinkage. The theoretical stresses in simple reinforced concrete members due to the shrinkage of the concrete are readily calculated upon the usual assumptions of perfect bond and constant modulus of

elasticity. However, the effect of plastic flow upon the stresses and deformations caused by shrinkage of the concrete is complicated by the fact that the stress is not constant over the period during which shrinkage occurs, and therefore the plastic flow is less than it would be under a uniform sustained stress. To offset this partially, the major portion of the shrinkage occurs at early ages (first 30 days) when the rate of plastic flow is greatest.

Glanville has shown[15] that the effect of plastic flow upon the stresses in columns under sustained load may be determined with sufficient accuracy, considering the vagueness of our knowledge of the plastic properties of the concrete, by the use of a reduced modulus of elasticity obtained from equation (4).

$$E_{cs} = \frac{E_c}{1 + E_c y} \qquad (4)$$

in which

E_{cs} = the effective modulus of elasticity
E_c = Instantaneous modulus of elasticity
y = plastic flow deformation for the time under consideration (perhaps 90 days) obtained from equation (1), (2) or (3).

If this reduced modulus of elasticity be used in connection with the usual theory for stresses due to shrinkage,* the results will be as accurate as our present knowledge of the materials in service warrants. For ordinary concretes cured in air under average conditions, this will give an effective modulus of elasticity for concrete of about 1,000,000 p.s.i.

Stresses, Sustained Load. Here also the uncertainty as to the physical properties and external conditions surrounding the concrete in the structure renders the use of complicated theory as to stress distribution unjustifiable. The stresses due to shrinkage, dead load, and any permanent load should be calculated by the use of a reduced modulus of elasticity as given in equation (4). To these stresses should be added those produced by temperature variations and live load, calculated by the use of the usual instantaneous modulus of elasticity. This involves the use of two locations for the neutral axis of members in flexure which is probably inconsistent but no great error can result. In calculating this reduced modulus, the ultimate value of y for plastic flow should be used (after 5 years). For ordinary concretes in air, this may cause the effective modulus of elasticity to fall well below 1,000,000 p.s.i.[18], [19].

It will be found for columns and for beams reinforced for compression, that the compressive stresses in the reinforcing steel are very

*"Principles of Reinforced Concrete Construction" Turneaure and Maurer, Fourth Edition, p. 185.

high[18], [19], in some instances approaching the elastic limit of the steel. It is believed that these high steel stresses are not dangerous, so long as the steel is well tied to prevent lateral deflection or buckling. Care must be exercised in designing splices for the compression reinforcement when these high unit stresses exist.

Deformations, Sustained Load. As both the elastic and plastic deformations are taken as proportional to stress, the virtual effect of plastic flow upon the deflection of beams or columns is to change the ratio of the moduli of elasticity. The plastic flow added to the elastic deformation reduces the effective concrete modulus, as given by equation (4). This changed modulus of the concrete will change the unit stresses in both the concrete and the steel. The concrete unit stress will be reduced and the steel unit stress increased.

In computing deflections of beams the "cracked" section should not be used because the deflection depends upon the deformations for the entire length of the beam and the proportion of this length covered by crack openings is very small. It is usually regarded as sufficiently accurate to disregard the concrete below the tension steel and to use the moment of inertia of the transformed section. Then the usual deflection formulas for homogenious beams may be used. It should be noted that the changed value of the ratio of the moduli of elasticity (n) causes a considerable movement of the neutral axis of the beam, in calculating both stresses and deflections.

In calculating deflections and using the entire concrete section above the tension steel as effective, the location of the neutral axis is obtained from

$$k = \frac{\frac{1}{2} + n\left(p + \frac{d'p'}{d}\right)}{1 + n(p + p')}$$

in which p' = percentage of reinforcing steel in the compression area,
d' = distance from the compression face to the compression steel.

If no compression steel is used,

$$k = \frac{\frac{1}{2} + np}{1 + np}$$

The shortening of columns and the deflections of beams due to shrinkage, plastic flow and sustained loads will be found to be greatly in excess of the instantaneous deformations, and may be a controlling factor in the design of some structures.

Discussion of the foregoing report will be welcome if received in triplicate by the Secretary of the Institute by February 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for March-April, 1937.*

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MAY-JUNE 1937

Discussion of a Report of Committee 313:

Effect of Plastic Flow and Volume Changes on Design*

CONVENTION DISCUSSION

A. S. Douglass†: Having talked with Professor Shank about this matter, the questions I shall ask are for a recorded answer. We are interested in a concrete material novel in some respects, consisting of cinders, fly ash and portland cement—our interest to dispose of a power industry by-product. We have not learned as much about the material as we wish to know. We suspect it will have more shrinkage in drying than standard pebble concrete and that it will have more plastic flow. Referring to the top of page 127°, it is *believed* that these high steel stresses are not dangerous as long as the steel is well tied. For a building of importance, that word "believed" is not too encouraging. If we find it true that this material has greater shrinkage and greater plastic flow than the pebble concrete, is it not possible that the stresses which this report indicates may be approaching the elastic limit, will then exceed the elastic limit, and in that event what design precaution is recommended?

Prof. J. R. Shank‡: That is a rather large question. If the material that Mr. Douglass speaks of has the large plastic flow as well as the large shrinkage that he suspects, the plastic flow may help to offset the effect of the shrinkage. If the plastic flow is large the increase in the steel stress is likely to be rather high. If the steel is carried beyond its elastic limit, it will have a plastic flow of its own. It is rather difficult to indicate what the stress in the steel may be when the stress is above the elastic limit. If the steel is rigidly held integrally with

*JOURNAL Amer. Concrete Inst. Nov.-Dec. 1936, *Proceedings* Vol. 33 p. 123. Presented at 33rd Annual Convention, New York, Feb. 23-26, 1937, by Prof. J. R. Shank, for Prof. Clyde T. Morris, Chairman Committee 313.

†Construction Engineer, The Detroit Edison Co., Detroit, Mich.

‡Ohio State University, Columbus, Ohio.

the member, this high stress, though not a desirable condition, is nevertheless not exceedingly dangerous. There are others here who are well able to amplify the answer to this question—Professor Richart and others.

F. E. Richart:* Speaking with reference to columns, it is certainly possible to have the longitudinal steel stressed beyond the yield point without serious danger, so long as buckling of the steel can be prevented. This is possible in an axially loaded spiral column; in tests to failure, the shortening may reach 10 times the amount that produces yield point stress. In a tied column or a beam with compression reinforcement the situation is not so reassuring, since dependence must be placed on ties or stirrups and on the tensile strength of the concrete to keep the compressive steel in line. However, if sufficient reinforcement is used to be effective in holding down the deformations, the chances of exceeding the yield point are greatly reduced. In tests of flexural members of gravel concrete under two years of sustained loading at computed working stresses, we found no serious stresses in compression steel when used in amounts of one per cent or more.

Mr. Douglass: I should like to ask two other questions. On page 125, I believe, in formula one, a should be expressed as a root: is that correct?

Professor Shank: The exponent a is incorrectly placed.

Mr. Douglass: Then I should like to ask what preference there is between medium steel and rail steel—not as a general controversial subject, (I notice your smile and I should not venture to open that question) but this is strictly specific in reference to the case in hand, referring to Professor Shank's reference to the plastic flow of steel. Would he prefer a medium steel more susceptible to plastic flow than a rail steel less susceptible?

Professor Shank: It appears that in a steel with a low elastic limit, say forty thousand p.s.i., if the plastic flow and shrinkage of the concrete should carry the stress in the steel beyond the elastic limit, the plastic flow in the steel would come into service and the redistribution of the internal stresses would not take place to the extent that calculations from deformation data by elastic theory might indicate. For a steel with a higher elastic limit the redistribution would be carried much farther and much higher stresses would result in the steel with consequent lower values in the stresses of the concrete. Whether that is to be considered to be good or bad is another question. The steel

*Research Professor of Engineering Materials, University of Illinois, Urbana.

with the higher elastic limit should certainly be more carefully tied in than the one with the lower, because the stresses will be higher and the tendency to buckle out is almost certain to be considerably greater.

*R. W. Crum:** Has there been any conclusion as to the variance of plastic flow according to the yield strength of the concrete?

Professor Shank: My recollection is that for the same load the lower grade concrete, that is, 2000-lb. concrete, would have a higher plastic flow than would a higher strength concrete.

Charles S. Whitney†: Dr. Glanville's tests indicated a lower flow for higher strength concrete. That is one of our serious problems now—in using more rapid setting cement. There is a tendency towards increasing the cracking in structures and I believe that it is due to the reduction of the plasticity of the concrete after setting on account of the high early strength cement.

Professor Richart: When the same load is applied to a weak concrete and to a strong one, it is obvious that there should be greater flow in the weaker concrete. However, it would seem fairer to load each kind of concrete to a certain fraction of its ultimate, since this is the basis of determining working stresses in building codes. In tests of plain columns of 2000-, 3500- and 5000- lb. gravel concrete loaded to one-fourth of the ultimate load for a year, we found respective values of shrinkage (in air of laboratory) of .00041, .00044 and .00041 in. per in., and corresponding values of combined flow and shrinkage of .00100, .00125 and .00125. This indicates somewhat greater flow for the richer concretes.

*Director Highway Research Board, Washington, D. C.
†Consulting Engineer, Milwaukee.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . NOVEMBER-DECEMBER 1936

A Conductometric Analysis of Portland Cement Pastes and Mortars and Some of Its Applications*

By W. B. Boast†

INTRODUCTION

Electrical conductometric analyses often afford relatively simple means of interpreting and measuring phenomena which may be impossible or difficult of interpretation or measurement by other methods. Such analyses have been applied to many industrial and manufacturing control processes, but only a very limited amount have dealt directly with portland cement pastes or mortars. The purpose of this investigation has been to develop conductometric methods for several problems related to portland cement pastes and mortars, make tests upon the substances, and present the results with what appear to be the logical interpretations.

From an electrical viewpoint a cement paste or mortar consists of a more or less granular substance saturated with an electrolytic solution or colloidal gel. Variations in the ratio of this solution or gel to the granular substance, and variations in the concentration and viscosity effects in the electrolyte produce changes in the electrical conductivity. From these changes deductions may be drawn as to the physical processes which occur in the paste or mortar.

The problems which have been investigated include: (1) the setting phenomena during the first three hours after gauging a cement paste; (2) the excess water tendency in sand-water mixtures; (3) measurements of internal stratification effects in cement pastes (i. e., the relative vertical movement of water, cement particles, and other constituents of a plastic mass as caused by capillary and gravitational forces); and (4) an attempted correlation of the electrical conductivity and the 28-day compressive strength of mortar mixes.

*Abstract of a doctoral thesis of the same title by W. B. Boast, Iowa State College, August, 1936. Received by the Institute Secretary, Oct. 26, 1936.

†Instructor, Dept. of Electrical Engineering, Iowa State College, Ames, Ia.

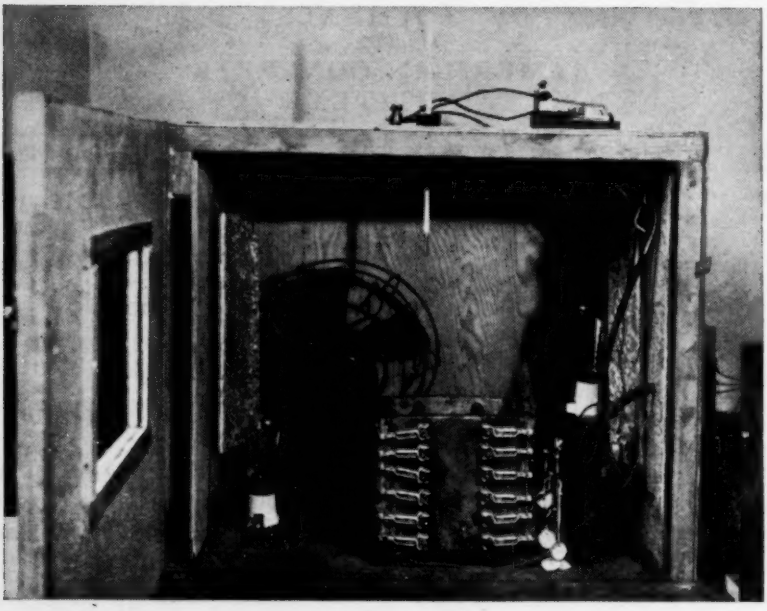


FIG. 1—PHOTOGRAPH OF TESTING CABINET WITH CONTAINER FOR STRATIFICATION MEASUREMENTS IN PLACE

APPARATUS

The chief items of apparatus are briefly described under the following paragraph headings.

Oscillator. Electrical power for all conductivity measurements was obtained from a 1000-cycle vacuum-tube oscillator.

Sample containers for conductivity measurements. The sample containers for the cement pastes and mortars consisted of paraffined wooden containers having approximate inside dimensions of 12.5 by 2.5 by 2.5 cm. Molded graphite carbon brushes cut to fit the 2.5 by 2.5 dimensions of the box were used for electrodes.

The container used for the sand-water mixtures was of the same type. Its dimensions were 26.0 by 6.4 by 5.1 cm.

Sample container for stratification measurements. The container for stratification measurements is shown in Fig. 1. It consisted of a paraffined wooden box with paraffin-sealed joints. The dimensions of the box were: height, 15.2 cm.; width, 10.2 cm.; and length, 15.2 cm. The electrodes consisted of twelve graphite carbons 1.1 cm. in diameter and approximately 18 cm. in length. The arrangement of these electrodes is evident in the photograph. Connections to them were made with test clips.

The electrical circuit used in making these determinations of stratification is shown in Fig. 2.

Wheatstone bridge apparatus. The modified Wheatstone bridge circuit is shown in Fig. 3. The condensers were necessary to balance the polarization effects at the sample-electrode surfaces.

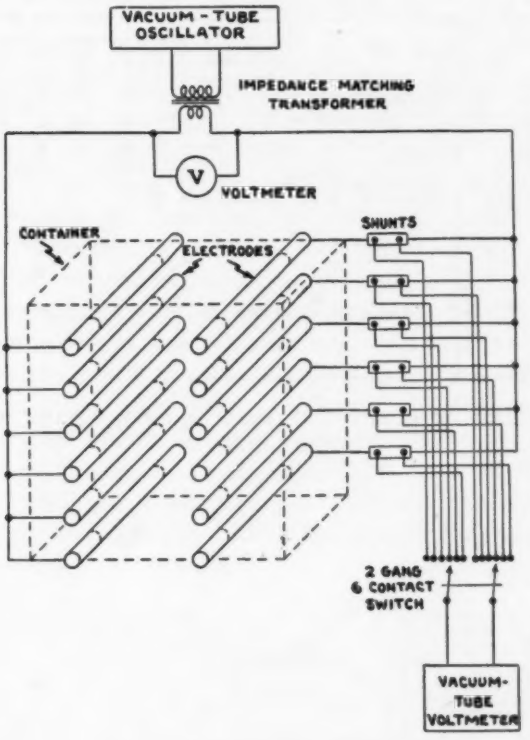


FIG. 2—SCHEMATIC DIAGRAM OF ELECTRICAL CIRCUIT FOR STRATIFICATION MEASUREMENTS

Vacuum-tube voltmeter. A vacuum-tube voltmeter was used for measuring the voltage drops across the shunts for the stratification measurements (Fig. 2). The voltmeter consisted of a two-stage amplifier and detector This voltmeter, as well as the oscillator and the bridge equipment, is evident in Fig. 4.

Electrical testing cabinet. The electrical testing cabinet of double-wall construction was thermostatically controlled for holding the ambient temperature of the sample containers at the desired level. The cabinet is shown in Fig. 1.

Curing cabinet. The water used for curing the compressive strength samples was maintained at constant temperature by thermostatic control. Sufficient tap water was allowed to flow into the box to maintain the temperature several degrees below that desired. The water was then boosted to the desired temperature by a controlled immersed heating element.

Compression testing machine. A 75,000-lb. compression testing machine was used for making compressive strength tests on the cured mortar cubes.

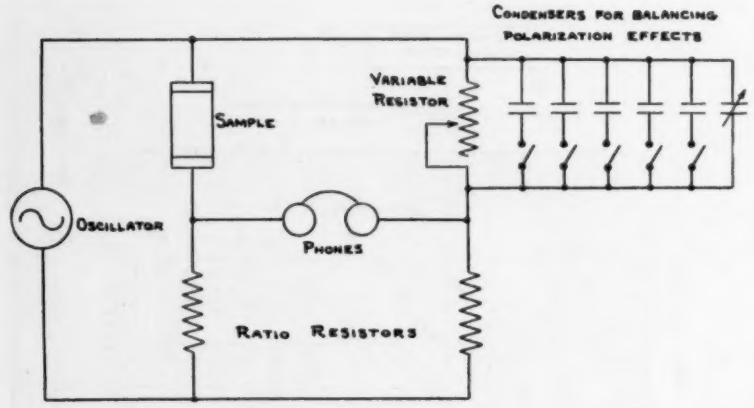


FIG. 3—DIAGRAM OF ELECTRICAL CIRCUIT OF MODIFIED WHEATSTONE BRIDGE CIRCUIT

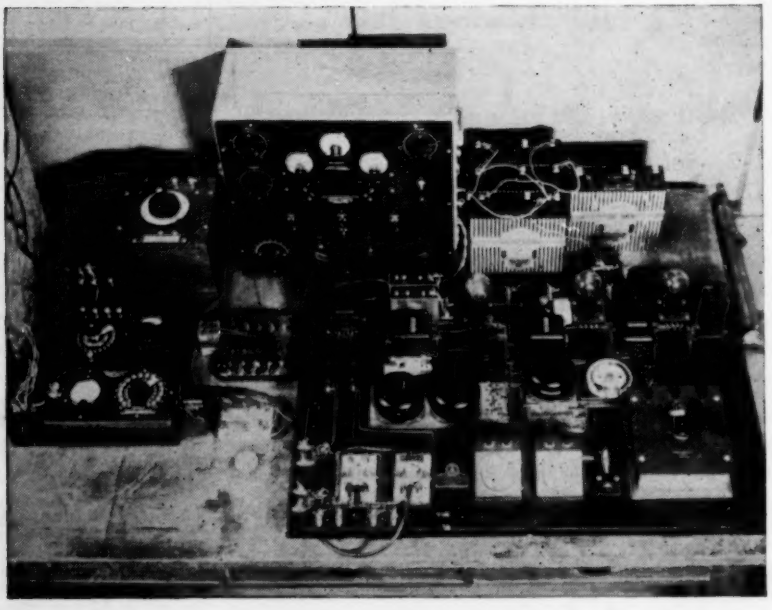


FIG. 4—PHOTOGRAPH OF WHEATSTONE BRIDGE EQUIPMENT, OSCILLATOR, AND VACUUM-TUBE VOLTMETER

THE INVESTIGATIONS

Setting Phenomena of Cement Pastes

The setting phenomena for cement pastes of three brands of portland cement, at water-cement ratios ranging from 0.25 to 0.40 by weight and at temperatures from 22° to 30° C., were followed conductometrically for three hours. Four or five samples at each water-cement ratio were necessary to determine the variation of conductivity with temperature. The results of these tests were plotted in two sets of curves. In each set the temperature was plotted as the abscissa and the electrical conductivity as the ordinate. One set of curves was plotted at increments of water-cement ratio, the other at increments of time. These two sets of families of curves were then checked against each other for inaccuracies in the estimations of the smooth curves. From the original plots, values of conductivity were tabulated at temperatures of 22, 24, 26, 28, and 30° C., at times of 15, 30, 45, 60, 90, 120, 150, and 180 minutes, and at water-cement ratios of 0.25, 0.30, 0.35, and 0.40 by weight. These tabulations for one of the brands of cement are made in Table 1.

TABLE 1—SUMMARY OF VARIATION OF CONDUCTIVITY WITH TEMPERATURE, WATER-CEMENT RATIO, AND TIME FOR CEMENT B

Water / Cement	Time (Min.)	Conductivity (10^{-3} mhos/cm^3) at Temperature of				
		22° C.	24° C.	26° C.	28° C.	30° C.
0.250	15	13.4	13.8	14.1	14.4	14.6
	30	14.0	14.4	14.8	15.1	15.4
	45	14.4	14.8	15.2	15.6	15.9
	60	14.6	15.1	15.6	16.1	16.5
	90	14.5	14.9	15.3	15.5	15.8
	120	14.3	14.6	14.9	15.2	15.4
	150	13.9	14.1	14.3	14.5	14.6
	180	13.3	13.4	13.5	13.6	13.7
0.300	15	13.7	14.2	14.7	15.2	15.6
	30	14.3	15.0	15.6	16.1	16.6
	45	14.8	15.4	16.0	16.6	17.1
	60	15.1	15.8	16.4	17.1	17.7
	90	15.1	15.6	16.1	16.6	17.0
	120	14.8	15.3	15.8	16.2	16.6
	150	14.4	14.9	15.3	15.6	15.9
	180	14.0	14.6	14.8	15.0	15.1
0.350	15	13.9	14.5	15.1	15.6	16.1
	30	14.6	15.3	15.9	16.5	17.1
	45	15.0	15.7	16.3	17.0	17.5
	60	15.3	16.0	16.7	17.3	17.9
	90	15.3	15.9	16.5	17.0	17.5
	120	15.0	15.6	16.1	16.5	16.9
	150	14.8	15.2	16.1	16.5	16.9
	180	14.5	14.8	15.1	15.4	15.7
0.400	15	14.1	14.8	15.4	16.0	16.6
	30	14.8	15.5	16.2	16.8	17.4
	45	15.2	15.9	16.6	17.2	17.8
	60	15.5	16.2	16.9	17.5	18.1
	90	15.6	16.2	16.7	17.3	17.8
	120	15.3	15.8	16.3	16.8	17.2
	150	15.0	15.4	15.8	16.1	16.5
	180	14.7	15.0	15.3	15.6	15.9

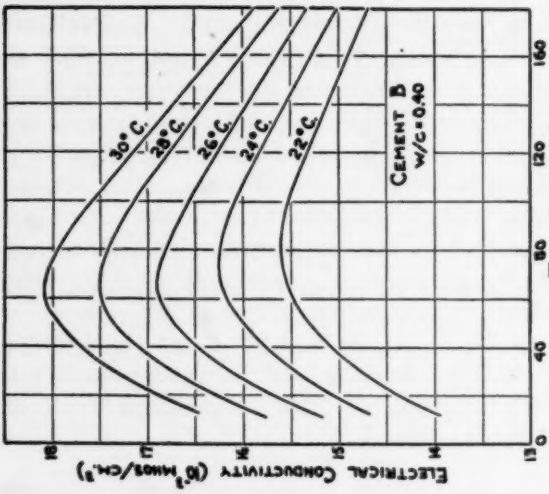


FIG. 6—EFFECT OF TEMPERATURE UPON THE SETTING CHARACTERISTICS

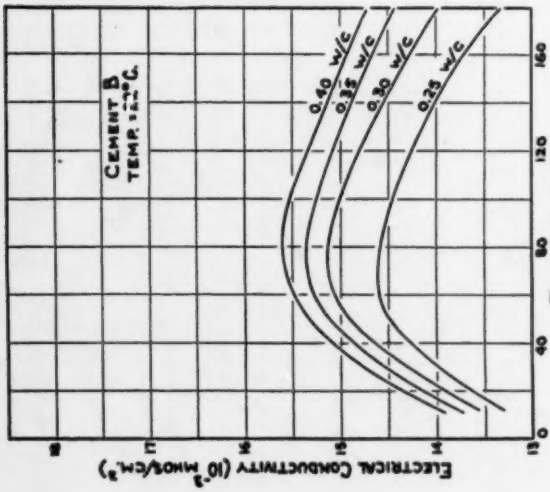


FIG. 5—EFFECT OF WATER-CEMENT RATIO UPON THE SETTING CHARACTERISTICS

At constant water-cement ratio and temperature the electrical conductivity was found to increase for a period, reach a maximum, and then decrease. The value of time at which this maximum conductivity was reached was influenced by both the water-cement ratio and the temperature. Several typical setting characteristics are shown in Fig. 5 and 6.

The form of the conductivity-time curve may be explained as follows:

There are two distinctly different phenomena which occur in the setting of cement pastes. That which is predominant in the early period after gauging is a hydration process. The increased hydration of the surface of the cement particle causes an increase in the ionization of the electrolyte and an increase in the effective cross sectional area of the sample available to current flow, and hence an increase in the electrical conductivity. That this hydration does not occur instantly is evident from the electrical measurements since the maximum conductivity is not reached until approximately an hour after gauging. The reaction evidently gradually works inwardly on each particle, and builds up a hydrated layer in the outer part of each.

The second phenomenon in the setting process is the formation of a colloidal gel and at later values of time perhaps the crystallization of certain constituents of the paste. The formation of this colloidal gel produces increased viscosity effects of the electrolyte and thereby decreases ionic mobility. Consequently conductivity decreases.

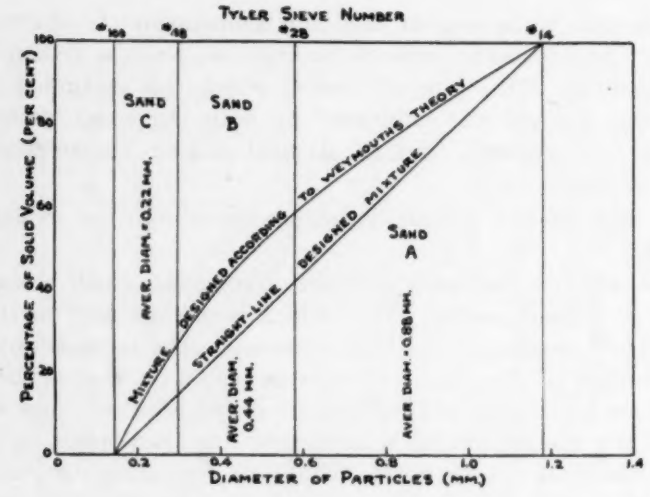
No doubt these two phenomena overlap more or less, but the net result is a maximum in the conductivity-time curve, which gives a measure of the periods over which the two processes predominate.

Decreased water-cement ratio and increased temperature both cause the maximum point to be reached more quickly. They also cause a larger negative slope in the conductivity-time curve during the colloidal-formation period. This is to be expected since the reactions occur more rapidly at higher temperatures or at lower water contents.

Excess Water Tendency in Sand-Water Mixtures

Sand-water mixtures were used in these tests because the sand particles were very inert electrically. The conductance of the mix was assumed, as a very good approximation, to be caused entirely by the electrolyte. The electrolyte consisted of tap water at 25°. C

The dry voids of several sizes of Ottawa silica sand were estimated by measuring the volumes of dry-rodded sand, a given quantity of water, and the volume of the combination after the sand was poured into the water. This measure is indicated by V_{dry} in Fig. 8.



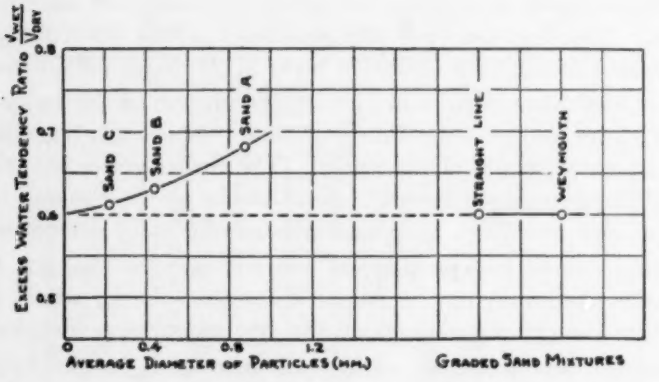


FIG. 7 (ABOVE) —SIZES AND GRADINGS OF SANDS USED FOR EXCESS WATER DETERMINATIONS

FIG. 8 (BELOW) RESULTS OF DETERMINATIONS OF EXCESS WATER TENDENCY

The conductivity of the tap water at 25° C. was then determined. Using the same sizes of sand as for the dry void measurements, each was in turn compacted in the water and readings of the conductivity repeated. The ratio of the conductivity of the sand-water mixture to that of the water alone gives a measure of the wet voids of the mixture. This measure is indicated by V_{wet} in Fig. 8.

This ratio does not give the exact percentage of wet voids for two reasons. First, the current lines through the sand voids, i.e., through the water, are not straight lines from one electrode to the other, and

second, the cross sectional areas of water through which the current lines pass are not uniform in dimensions and hence are not used most effectively. Both of these factors tend to make the ratio less than the actual percentage of wet voids. However, the ratio does give a relative measure of the wet voids.

If the proportionality between this relative measure of the wet voids and the measure of the dry voids were constant for all sizes of sand, there would be no indication as to whether one size of sand required an excess of water or not. That is, the measure gives only a relative indication. The author has defined the "excess water tendency," then, as the ratio of the conductometric measure of wet voids to the dry voids measured by the forenamed method.

Measurements were made also upon two gradings of sand mixes. These gradings included a straight-line grading and a mixture designed according to the work of C. A. G. Weymouth.* Both of these gradings exhibited an excess water tendency of approximately the same value as that for the extremely small sizes of sand particles.

The proportions of the sands used in the gradings are shown in Fig. 7. The results of the determinations are shown in Fig. 8.

Internal Stratification in Cement Pastes

The method used for determining the change in water content in lateral sections of a sample consisted essentially of measuring the electrical conductances of those sections of the sample. A diagram of the electrical circuit is shown in Fig. 2.

The sample container used for these measurements consisted of a container of 15.2 cm. depth in which electrode pairs were placed at several levels throughout the vertical height of the sample. By maintaining the same voltage on all the electrode pairs, the sample was sectionalized electrically into lateral layers. The currents flowing through each of the layers were determined from voltage-drop measurements, using a vacuum-tube voltmeter across low resistance shunts placed in each electrode circuit.

Percentage conductances, based upon the average conductance of all sections except the top, were calculated from these measurements. The top section of the sample was excluded from the basis since the errors encountered in the measurements upon this section were invariably large. Surface boundary conditions accounted for these discrepancies. The average results of three determinations are shown in Fig. 9 and 10. The data presented are for one of the cements at water-cement ratios of 0.35 and 0.40 by weight. The water which

*Powers, T. C. *A Discussion of C. A. G. Weymouth's Theory of Interference.* Research Laboratory of the Portland Cement Association, Chicago, 1936.

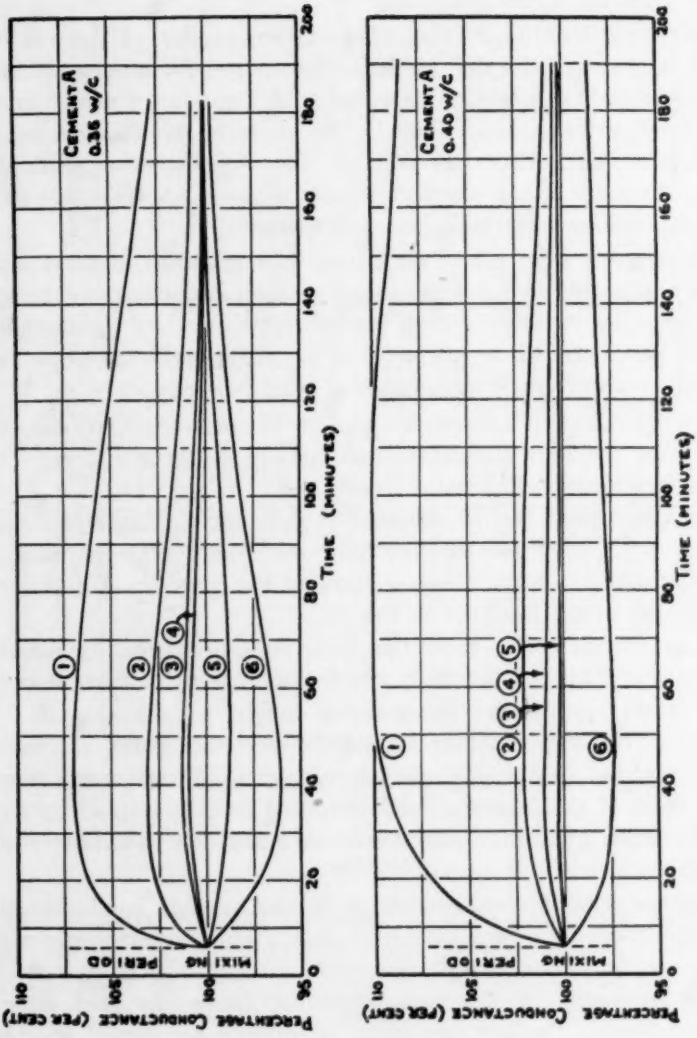


FIG. 9 (ABOVE) AND 10 (BELOW)—STRATIFICATION MEASUREMENTS UPON CEMENT A AT TWO WATER-CEMENT RATIOS

progressed upward through the sample during the early period of the test, as evidenced by the increased conductances nearer the top, was absorbed rather uniformly back into the sample for the 0.35 water-cement ratio sample. At 0.40 water-cement ratio the redistribution was very poor at 3 hours after gauging. The maximum spread in water differences occurred at a much earlier time for the 0.35 than for the 0.40 water-cement ratio sample.

Fig. 9 and 11 illustrate the stratification phenomena as it occurred in two cement pastes at the same water-cement ratio by weight. Cement A resulted in a much better redistribution of water content at 3 hours than did cement C. This method thus gives definite indications both of water progression upward and of the redistribution of water in that important period when hardening begins. It is because of the possible significant influences of this early stratification upon the final properties of the concrete that such a detailed method of test was devised and these particular tests performed.

Correlation of Electrical Conductivity and Compressive Strength of Cement Mortars

Mortar mixes of various proportions of sand, cement, and water were measured for electrical conductivity at 15 minutes after gauging, and for compressive strength at 28 days. Standard 20/30 Ottawa silica sand was used for all mortar mixes. Compressive strength tests were made upon 2-in. cubes.

Conductivity and compressive strength of cement mortars can best be compared by means of intermediary factors, either the water-cement ratio or the sand-cement ratio. For this reason, the results are plotted as four curves, Fig. 12, 13, 14, and 15. These curves are so grouped that the conductivity and strength can be compared at constant water-cement or sand-cement ratios.

The original purpose of this portion of the investigation was to devise a means whereby the compressive strength of a mortar mix could be predicted from the conductivity measurements. That this general correlation is impossible may be noted by referring to the figures. At constant sand-cement ratio an increase in conductivity corresponds to a *decrease* in compressive strength at workable mixes (to the right of the water-cement ratio producing the maximum strength). Whereas, for constant water-cement ratio an increase in conductivity corresponds to an *increase* in compressive strength. Thus, from a proper choice of mixes the same conductivity could correspond within limits to any compressive strength.

Although a complete correlation was thus found impossible, a very interesting relationship was observed when the sand-cement

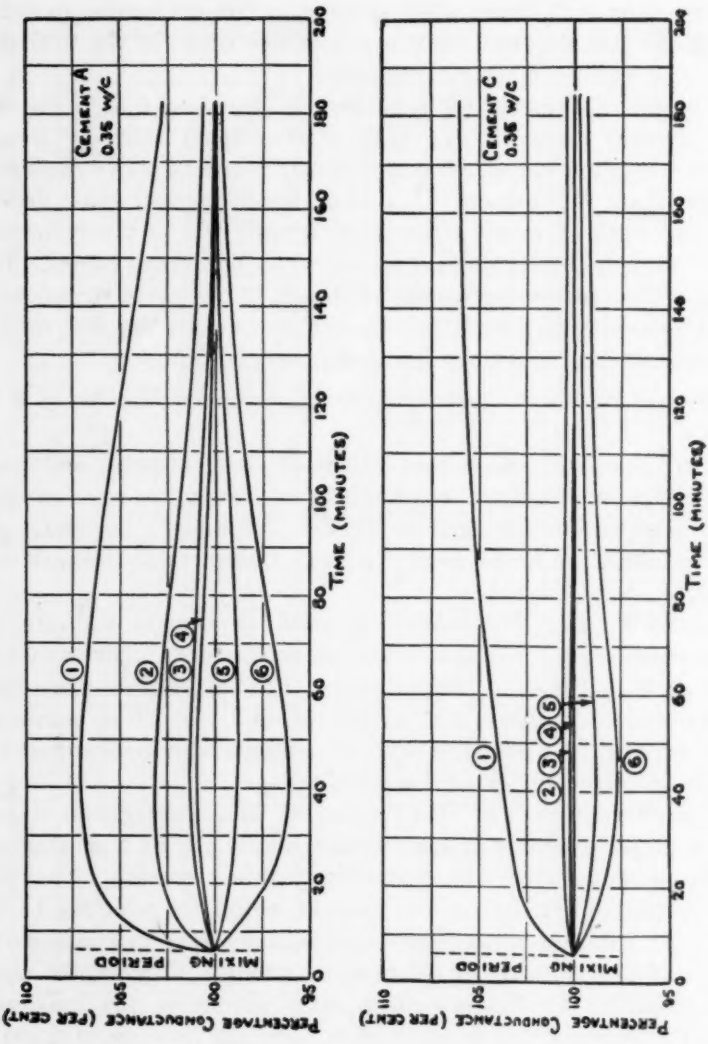
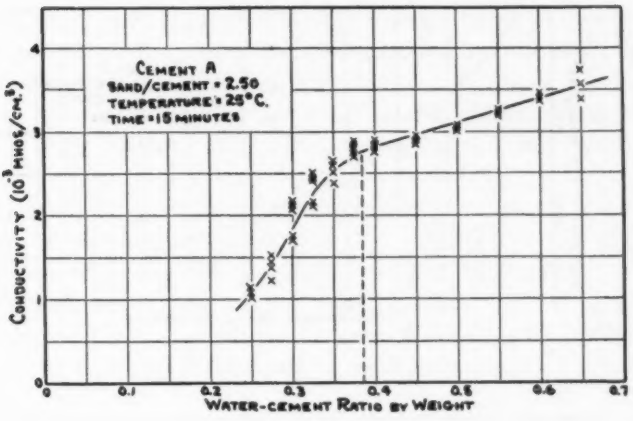


FIG. 9 (ABOVE, REPEATED) AND 11 (BELOW)—STRATIFICATION MEASUREMENTS UPON CEMENTS A AND C AT THE SAME WATER-CEMENT RATIO



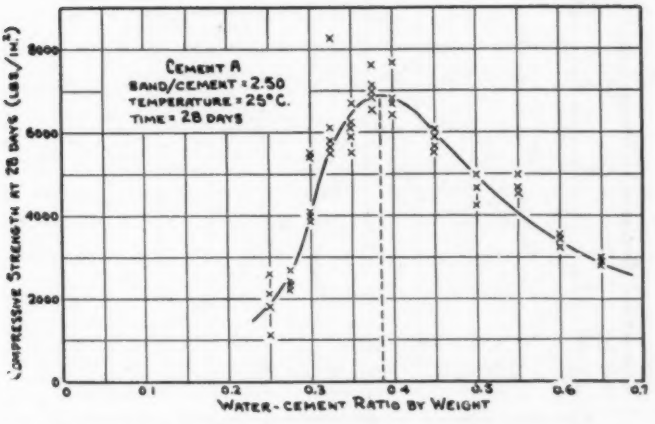
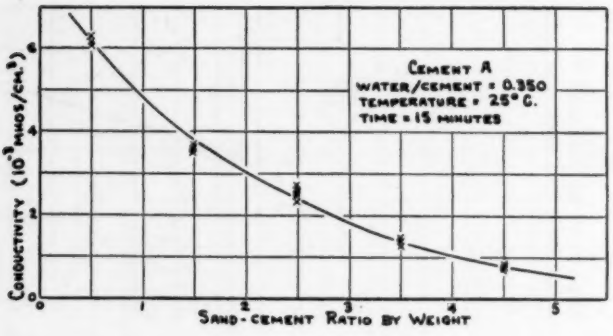


FIG. 12 (ABOVE)—ELECTRICAL CONDUCTIVITY AS A FUNCTION OF WATER-CEMENT RATIO

FIG. 13 (BELOW)—COMPRESSIVE STRENGTH AS A FUNCTION OF WATER-CEMENT RATIO

ratio was held constant (Fig. 12 and 13). The maximum compressive strength was found to occur at the same water-cement ratio as did a decided break or knee in the electrical conductivity curve. This seems to indicate that the most desirable water content in a cement mortar is that content which will allow the particles of mortar to be as close together as possible and yet completely saturate the voids of the mixture. At lower water contents the increase in air voids no doubt causes the sudden decrease in electrical conductivity.



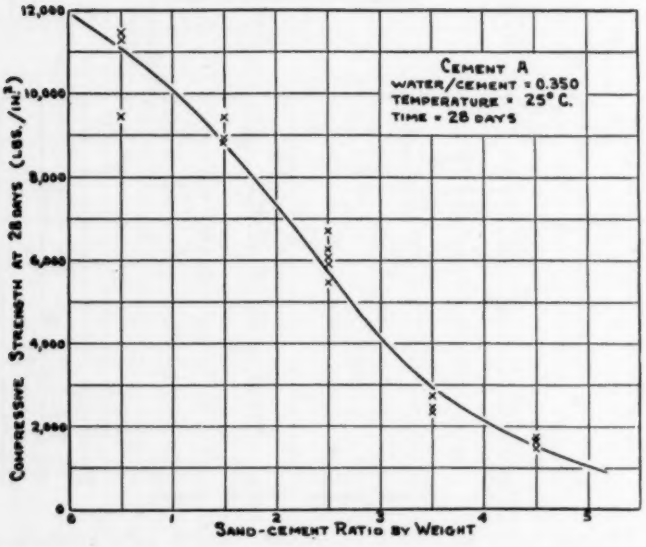


FIG. 14 (ABOVE)—ELECTRICAL CONDUCTIVITY AS A FUNCTION OF SAND-CEMENT RATIO

FIG. 15 (BELOW)—COMPRESSIVE STRENGTH AS A FUNCTION OF SAND-CEMENT RATIO

CONCLUSIONS AND SUMMARY

Setting Phenomena of Cement Pastes

Two phenomena which are evident from changes in the electrical conductivity occur in the setting of cement pastes. The first, a hydration process, causes increased ionization of the liquid portion of the paste and results in increased electrical conductivity with time.

The second phenomenon, a colloidal-formation process, causes increased viscosity effects in the electrically conducting media and results in decreased electrical conductivity with time.

The two processes no doubt overlap causing a maximum point in the conductivity curve. The time at which this maximum occurs gives a relative measure of the hydration period.

The negative slope of the conductivity-time curve gives a measure of the rate of colloidal formation.

An increase in temperature causes a shorter hydration period and more rapid colloidal formation for most cements.

An increase in water-cement ratio causes a longer hydration period and less rapid colloidal formation for most cements.

Excess Water Tendency in Sand-Water Mixtures

The excess water tendency (a measure of the excess water above that required to fill the dry voids of a granular material) decreases with decreased diameter of the particles.

Properly graded material has an excess water tendency of approximately the same amount as material which is all of very small diameter.

Internal Stratification in Cement Pastes

Although percentage conductance does not always bear an exactly linear relationship to percentage water content, the method of testing by conductance measurements gives a very positive indication of the degree of water rise and of the redistribution of water in cement pastes. The method can be applied equally well to cement mortars and cement pastes.

Correlation of Electrical Conductivity and Compressive Strength of Cement Mortars

A break in the conductivity versus water-cement ratio curve, at constant sand-cement ratio, occurs where the water content is just sufficient to fill the voids of the sample. Above this point the conductivity increases slowly with increased water-cement ratio. Below the point the conductivity decreases more rapidly with decreased water-cement ratio.

The compressive strength, at constant sand-cement ratio, is affected by the same requirement of sufficient water content as is the conductivity relationship. The optimum in compressive strength can be obtained when the water is just sufficient to fill the voids, but not in excess to produce spreading of the mortar particles.

At constant sand-cement ratio an increase in electrical conductivity corresponds to a decrease in compressive strength in the working range.

At constant water-cement ratio an increase in electrical conductivity corresponds to an increase in compressive strength.

A correlation between electrical conductivity and compressive strength for mortar mixes in general is impossible.

SELECTED REFERENCES

1. Britton, Herbert T. S. "Conductometric Analysis." Chapman and Hall, Ltd., London. 1934.

2. Brown, Levi S. "Bleeding of Cements." *Industrial and Engineering Chemistry*, Vol. 27, pp. 97-102. 1935.

3. Glasstone, S. "The Electrochemistry of Solutions." Methuen and Co., Ltd., London. 1930.

4. Jesser, Leopold. "Die Zementhärtung—Ein Elektrostatisches Phanomena II." *Zement*, Vol. 23, pp. 665-669, 677-682, 691-694. 1934.

5. Newman, F. H. "Electrolytic Conduction." John Wiley and Sons, Inc., New York. 1931.

6. Powers, T. C. "A Discussion of C. A. G. Weymouth's Theory of Particle Interference" Research Laboratory of the Portland Cement Association, Chicago. 1936.

7. Shimizu, Yosomatsu. "An Electrical Method for Measuring the Setting Time of Portland Cement." Tohoku Imperial University, Science Reports, First Series, Vol. 17, p. 85-92. 1928.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by February 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for March-April, 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . NOVEMBER-DECEMBER, 1936

A Short Method for Computing Moments in Continuous Frames*

By S. C. Hollister†
MEMBER AMERICAN CONCRETE INSTITUTE

INTRODUCTORY

When the several members of a building frame are firmly connected to each other at the joints, stresses are set up in the members by live loads on adjacent members, as well as by loads upon the members themselves. Unequal loads on adjacent spans, or equal loads on unequal adjacent spans, or some spans loaded with others not loaded, will greatly affect moments in the spans and will produce moments in intermediate columns. These moments, arising from action of the frame as a whole, are not determined by the usual moment coefficients. This paper furnishes a ready method for their approximate determination. It is based on the fundamental method of slope-deflections.‡

This method is presented, not in an effort to place on a too-crowded record "another method," but with a desire to simplify the computation of continuous frames to the greatest possible extent, compatible with required limits of accuracy. Only through such simplification will the effects of continuity receive widespread consideration.

NOTATION

The standard notation of slope-deflection will be used throughout. Thus (see Fig. 1), M_{AB} is the moment acting at the end A of the member AB; Θ_A is the rotation in radians of the joint A of the frame (hence, the rotation due to the given loading, of the tangent at the end A of all members meeting at A); K is the *stiffness ratio* I/l for any member, the member designated by a subscript; C_{AB} is the value of the moment M_{AB} *if* AB *were fixed*, and hereafter called the "load

*Received by the Institute Secretary Dec. 8, 1936.
†Associate Dean of College of Engineering, Cornell University, Ithaca, N. Y.
‡Presented originally in this country by G. A. Maney, Bull. 1, Univ. of Minn., 1915. See standard texts on structural analysis.

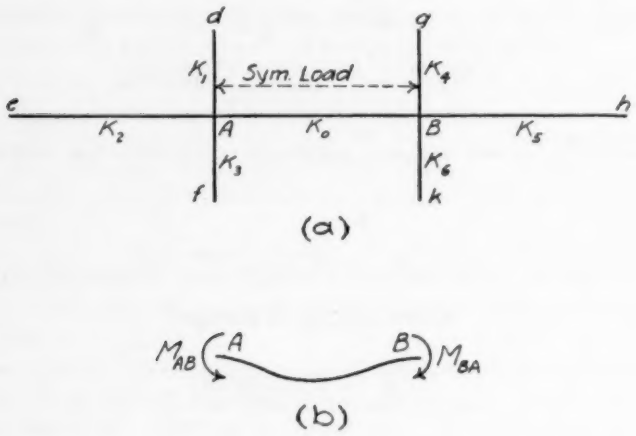


FIG. 1

term;" E is Young's modulus of elasticity for the material of which the member is made (considered constant throughout all members of a given frame of the same material).

SCHEME OF SIGNS

The sign of Θ is positive when Θ represents a clockwise rotation. The moment couple (as M_{AB}, likewise C_{AB}) is positive when it acts in a clockwise direction. It should be noted that M_{AB} (Fig. 1) or C_{AB} is the moment that must be applied to the member if it is cut free of the frame, in order to simulate the restraining effect of the support on the member at A. In Fig. 1(b), M_{AB} is negative (acting counter-clockwise) and M_{BA} is positive (acting clockwise).

SYMMETRICAL LOAD ON ONE SPAN

Let Fig 1(a) represent a portion of a building frame (hereafter such a portion of a frame will be called a "unit frame"), one span AB of the floor system being loaded by a symmetrically placed load.

The effect of this load is to cause the joints A and B to rotate, the amount of which rotation depends upon the stiffness of the members meeting at the joints. The members Ad, Ae, Af, Bg, Bh, Bk, are *restraining* members. Let Θ_A be the rotation produced at A and Θ_B that produced at B. At A, Fig. 1(b), from the slope-deflection theory,

$$\left.\begin{aligned} M_{AB} &= 2\,EK_o\,(2\Theta_A + \Theta_B) - C_{AB} \\ \text{and at } B \quad M_{BA} &= 2\,EK_o\,(2\Theta_B + \Theta_A) + C_{BA} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Similarly, from slope-deflection principles, the moment at the bottom of Ad is

$$M_{Ad} = 2EK_1\,(2\Theta_A + \Theta_d)$$

Now, if the restraining conditions at d are known, i.e., if the amount of rotation is known, Θ_d is known. Thus, if the column is fixed at d, $\Theta_d = 0$, and

$$M_{Ad} = 4\,EK_1\Theta_A \quad . \quad (\text{also, } M_{dA} = 2\,EK_1\,\Theta_A) \quad \ldots\ldots\ldots (2)$$

Likewise, if it is hinged at d, $M_{dA} = 0$; whence $\Theta_d = -\,\Theta_A/2$, and

$$M_{Ad} = 3\,EK_1\Theta_A \quad \ldots\ldots\ldots (3)$$

If $M_{Ad} = -M_{dA}$, $\Theta_d = -\,\Theta_A$ and

$$M_{Ad} = 2\,EK_1\Theta_A \quad \ldots\ldots\ldots (4)$$

If $M_{Ad} = M_{dA}$, $\Theta_d = \Theta_A$ and

$$M_{Ad} = 6\,EK_1\Theta_A \quad \ldots\ldots\ldots (5)$$

These four cases illustrate that in general the moment at A can be written

$$M_{Ad} = n_d\,EK_1\Theta_A \quad \ldots\ldots\ldots (6)$$

where n_d is a coefficient by which the degree of restraint (measured in terms of rotation) at d may be taken into account. For each restraining member, meeting either at A or B, eq. (6) gives the moment at the inner joint.

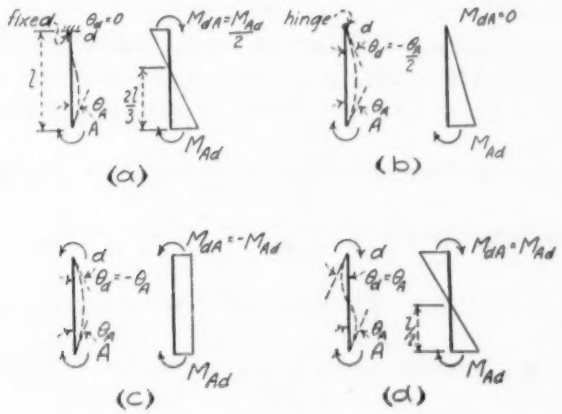


FIG. 2

The physical interpretations of eqs. (2) to (5) are important. They are shown in order in Fig. 2, together with the moment diagram for each. The smallest value of n_d is for eq. (4) when $n_d = 2$, shown in Fig. 2 (c) to represent a member having a constant moment over its entire length. When $n_d = 3$, (eq. (3) and Fig. 2 (b)), the moment at d is zero, indicating a hinge at d, and there is a uniformly increasing moment from d to A. When $n_d = 4$ (eq. (2) and Fig. 2 (a)), there is a fully fixed condition at d, and the point of inflection of the elastic line (hence, also the point of zero moment) for Ad is located $2l/3$

above A. When $n_d = 6$, (eq. (5) and Fig. 2 (d)), the point of inflection is at the mid-height of Ad.

It is significant that as the point of inflection moves from d (for which $n = 3$), down to the mid-point (for which $n = 6$), the value of n is increasing. Intermediate values of n, i.e., $n = 3.5$, $n = 5$, etc., represent corresponding intermediate locations of the point of inflection. It is, of course, possible to have conditions in which the point of inflection lies below the mid-point, in which case n is greater than 6. It is also possible to have conditions where n is less than 2, if the rotation at the far end of the member is increased by loads beyond, so that $\Theta_d > -\Theta_A$.

The occurrence of moments equivalent to Figs. 2(c) and 2(d) (hence also eqs. (4) and (5)) arise not from the load on AB, but from loads on the structure on the portion beyond d and thus not included in the "unit frame." It should now be clear that the designer may, by judicious selection of n_d, represent very closely the effect at d upon the unit frame of loads lying beyond. The application of this principle will be discussed later.

Since the summation of moments at the A-end of all members meeting at A equal zero, $\Sigma M_A = 0$. One obtains from eqs. (1) and (6),

$$4EK_o\Theta_A + 2EK_o\Theta_B - C_{AB} + n_d EK_1\Theta_A + n_e EK_2\Theta_A + n_f EK_3\Theta_A = 0$$

Or, rewriting, using the symbol $\Sigma^{3}_{1} nK$ to designate the summation of the nK terms for members 1 to 3,

$$\left(\Sigma^{3}_{1} nK + 4\,K_o\right) E\,\Theta_A + 2\,K_o E\,\Theta_B = C_{AB} \quad \ldots\ldots\ldots\ldots (7)$$

Similarly, taking moments about B, one obtains

$$2\,K_o E\Theta_A + \left(\Sigma^{6}_{4} nK + 4\,K_o\right) E\,\Theta_B = -C_{BA} \quad \ldots\ldots\ldots\ldots (8)$$

Eqs. (7) and (8) may be divided by K_o, so that for any member $K' = K/K_o$; then, dropping subscripts of C, since for the symmetrical load on AB, C_{AB} and C_{BA} are *numerically* equal,

$$\left(\Sigma^{3}_{1} nK' + 4\right) E\Theta_A + 2\,E\Theta_B = C/K_o \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

and

$$2\,E\Theta_A + \left(\Sigma^{6}_{4} nK' + 4\right) E\,\Theta_B = -C/K_o \quad \ldots\ldots\ldots\ldots\ldots (10)$$

Now, letting $a = \sum_{1}^{3} nK' + 4$ ..... (11)

and $b = \sum_{4}^{6} nK' + 4$ ..... (12)

one finds by solving eqs. (9) and (10) simultaneously,

$$E\,\Theta_A = \frac{b+2}{ab-4} \cdot \frac{C}{K_o} \quad \text{..... (13)}$$

and $$E\,\Theta = -\frac{a+2}{ab-4} \cdot \frac{C}{K_o} \quad \text{..... (14)}$$

Substituting these values into eqs. (1) one has finally

$$M_{AB} = \frac{-(a-4)(b+2)}{ab-4} C \quad \text{..... (15)}$$

$$M_{BA} = \frac{(b-4)(a+2)}{ab-4} C \quad \text{..... (16)}$$

The signs are such that the end couples (see Fig. 1(b)) cause a clockwise rotation when positive.

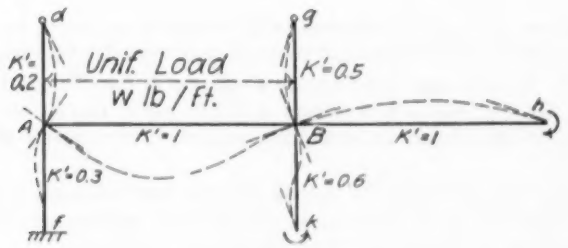


FIG. 3

Example 1. Suppose a frame to be as shown in Fig. 3 loaded on AB with w lb. per ft. Assume d and g hinged and f fixed; $M_{kB} = -M_{Bk}$ and $M_{kB} = M_{Bk}$. Then from eqs. (2) to (5) $n_d = n_g = 3$; $n_f = 4$; $n_k = 2$; $n_h = 6$.

$C = \frac{wl^2}{12}$; $a = (3\times0.2) + (4\times0.3) + 4 = 5.8$; $b = (3\times0.5) + (2\times1) + (6\times0.6) + 4 = 11.1$

Then $M_{AB} = -\frac{(5.8-4)(11.1+2)}{(5.8)(11.1)-4} \cdot \frac{wl^2}{12} = -\frac{23.58}{60.38} \cdot \frac{wl^2}{12} = -0.391\,\frac{wl^2}{12}$

Likewise, $M_{BA} = +\frac{(11.1-4)(5.8+2)}{(5.8)(11.1)-4} \cdot \frac{wl^2}{12} = +\frac{55.38}{60.38} \cdot \frac{wl^2}{12} = +0.917\,\frac{wl^2}{12}$

Now, at A it should be clear that since the moment is brought to the point A by the load on AB, the only opposition is in the resisting moments at A in the restraining members Ad and Af; and that, since $M_{Ad} + M_{Af} = M_{AB}$, for equilibrium, the reaction moment to M_{AB} is divided between Ad and Af in proportion to their respective nK' terms. Thus, the total restraint of Ad is $3 \times 0.2 = 0.6$; and of Af, is $4 \times 0.3 = 1.2$; hence, since the resisting moments are of opposite sign to M_{AB},

$$M_{Ad} = -\frac{0.6}{0.6 + 1.2} M_{AB} \text{ and } M_{Af} = -\frac{1.2}{0.6 + 1.2} M_{AB}$$

$$\text{or } M_{Ad} = +\frac{1}{3} \quad 0.391 \frac{wl^2}{12} = 0.130 \frac{wl^2}{12}; \text{ and } M_{Af} = +\frac{2}{3}\ 0.391 \frac{wl^2}{12} = 0.262 \frac{wl^2}{12}$$

Similarly, at B

$$M_{Bf} = -\frac{1.5}{7.1} \times 0.917 \frac{wl^2}{12} = -0.194 \frac{wl^2}{12}$$

$$M_{Bh} = -\frac{2.0}{7.1} \times 0.917 \frac{wl^2}{12} = -0.258 \frac{wl^2}{12}$$

$$M_{Bk} = -\frac{3.6}{7.1} \times 0.917 \frac{wl^2}{12} = -0.465 \frac{wl^2}{12}$$

The moment $M_{fA} = \frac{1}{2} M_{Af}$ since f is fixed. (This may be verified from eqs. (1) by making C and Θ_f equal to zero).

The moment at the center of AB is

$$\frac{M_{AB} - M_{BA}}{2} + \frac{wl^2}{8} = \frac{-0.391 - 0.917}{2} \cdot \frac{wl^2}{12} + \frac{3}{2} \frac{wl^2}{12} = 0.846 \frac{wl^2}{12}$$

Fig. 4(a) shows the moment diagram, with moments expressed as coefficients of $wl^2/12$. Fig. 4(b) shows the scheme of moment signs applied to the separate members.

Example 2. A continuous beam having a constant K for all spans and with spans as shown in Fig. 5(a) carries a dead load of 100 lb. per ft. and a live load of 200 lb. per ft. It is desired to find the *maximum* dead and live moments at the supports B and C.

Consider the first unit frame to include only the first two spans with AB loaded. Since A is hinged, $a = 4$. Take $n_e = 3.5$, which is midway between hinged and fixed conditions at C. Then $b = 7.5$ whence $M_{BA} = 0.807C_1$ and $M_{BC} = -0.807C_1$. Now, for $n_e = 3.5$ it can be shown that $M_{CB} = \frac{-1}{3.5} M_{BC}$ (it must *not* be interpreted that from this $M_{CB} = -M_{BC}/n$); whence $M_{CB} = 0.231\ C_1$. The resulting moment diagram for the first unit frame is shown in Fig. 5(b).

The second unit frame will be taken to include the first three spans with BC loaded. Assuming $n_A = 3$ and $n_D = 3.5$, the diagram becomes that shown in Fig. 5(c).

The third and fourth unit frames are likewise taken with three spans at a time with the center one loaded. The values of n at all extremities are assumed to be 3.5. The resulting moment diagrams are shown in Figs. 5(d) and (e) in solid lines.

To obtain the dead load moment at B, combine algebraically the moments at B as shown in Figs. 5(b), (c) and (d). With $C_1 = 676$; $C_2 = 833$; $C_3 = 2135$ ft.-lb., the total dead load moment at B is 647 ft.-lb. from the three unit frames. Further refinement may be made by adding the effect at B of the load in the fourth span as

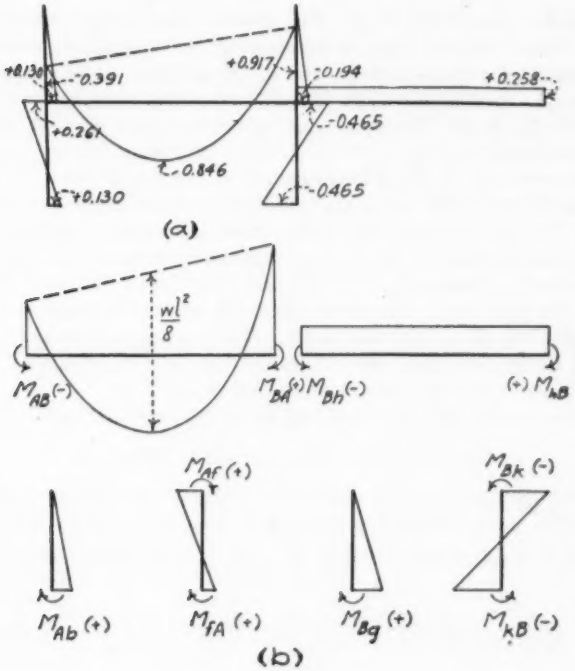

+0.130
-0.391
+0.917
-0.194
+0.258
+0.261
-0.846
-0.465
+0.130
-0.465
(a)
$\frac{wl^2}{8}$
M_{AB}(-)
M_{BA}(+) M_{Bh}(-)
(+) M_{kB}
M_{Af}(+)
M_{Bk}(-)
M_{Ab}(+)
M_{fA}(+)
M_{Bg}(+)
M_{kB}(-)
(b)

FIG. 4

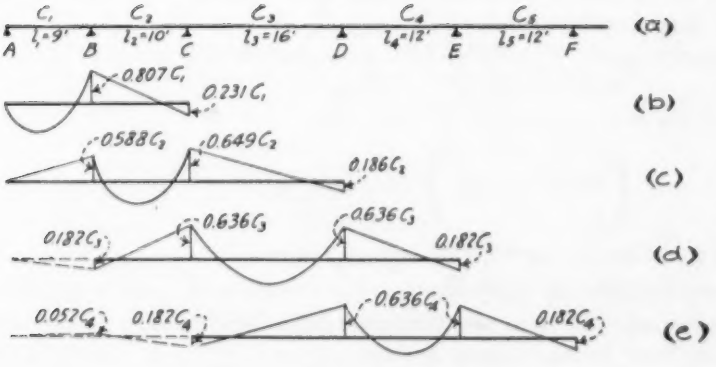

C_1 C_2 C_3 C_4 C_5
A l_1=9' B l_2=10' C l_3=16' D l_4=12' E l_5=12' F
(a)
0.807C_1
0.231C_1
(b)
0.588C_3
0.649C_2
0.186C_2
(c)
0.187C_3
0.636C_3
0.636C_3
0.187C_3
(d)
0.052C_4
0.182C_4
0.636C_4
0.182C_4
(e)

FIG. 5

shown by broken lines in Fig. 5(e). This effect is more marked in continuous beams on freely rotating supports than in beams attached to columns, since in the latter case the moment dissipates rapidly, due to the columns, as one moves away from the loaded span. In this case 62 ft.-lb. would be added, bringing the total dead load moment up to 709 ft.-lb. The true value by exact analysis is 740 ft.-lb., showing an error of 4.2 per cent.

In a similar manner the dead load moment at C is found from the first four unit frames to be 1524 ft.-lb., or with the added effect from the load on the fifth span, 1586 ft.-lb. The true value is 1566 ft.-lb., showing an error of 1.3 per cent.

The moment diagrams of Fig. 5 show that the maximum live load moment at B comes from loading spans one, two and four. C_1, C_2 and C_4 are double the values given above for dead load. The total maximum moment is found to be 2223 ft.-lb. This is within 5 ft.-lb. of the true value. The maximum live load moment at C is developed by loading spans two, three and five; and its computed value is 3920 ft.-lb., as compared with the true value of 3846 ft.-lb.

The total maximum dead and live load moments may now readily be found.

SPANS AND UNSYMMETRICAL LOADING

The foregoing eqs. (15) and (16) relate to loads symmetrically placed on the span, whether the span arrangement is symmetrical or not. If the load arrangement is not symmetrical these equations become

$$M_{AB} = \frac{-(a-4)(b\,C_{AB} + 2\,C_{BA})}{ab-4} \quad \ldots\ldots\ldots\ldots\ldots (17)$$

$$M_{BA} = \frac{(b-4)(aC_{BA} + 2\,C_{AB})}{ab-4} \quad \ldots\ldots\ldots\ldots\ldots (18)$$

in which C_{AB} and C_{BA} are respectively the moments at A and B *if these ends were fixed.*

LOADED MEMBER WITH FIXED END

Fig. 6 illustrates a common modification of the unit frame, in which the loaded span is fixed at one end. A modification of eqs. (15) to (18) will therefore be useful.

Applying eq. (1) to Fig. 6(a), we have when B is fixed,

$$M_{AB} = 4\,EK_o\Theta_A - C_{AB}$$

since $\Theta_B = O$. Then eq. (7) becomes

$$\begin{pmatrix} 3 \\ \Sigma nK + 4\,K_o \\ 1 \end{pmatrix} E\Theta_A = C_{AB}$$

and by substituting eq. (11) into the above,

$$E\Theta_A = C_{AB}/aK_o \quad \ldots\ldots\ldots\ldots\ldots (19)$$

Whence eq. (15) becomes, for *symmetrical load* on AB,

$$M_{AB} = -\frac{a-4}{a}\,C \quad \ldots\ldots (B \text{ fixed}) \quad \ldots\ldots\ldots (20)$$

Likewise, eq. (16) becomes

$$M_{BA} = +\frac{a+2}{a} C \ldots\ldots\ldots\ldots (B \text{ fixed}) \ldots\ldots\ldots (21)$$

When the load on AB is unsymmetrical, from the above,

$$M_{AB} = -\frac{a-4}{a} C_{AB} \ldots\ldots\ldots (B \text{ fixed}) \ldots\ldots\ldots (22)$$

$$M_{BA} = +a\frac{C_{BA} + 2C_{AB}}{a} \ldots\ldots (B \text{ fixed}) \ldots\ldots\ldots\ldots (23)$$

Similarly, for Fig. 6(b) in which A is fixed and $\Theta_A = O$, there results, for *symmetrical loading* on AB,

$$M_{AB} = -\frac{b+2}{b} C \ldots\ldots\ldots\ldots (A \text{ fixed}) \ldots\ldots\ldots (24)$$

$$M_{BA} = +\frac{b-4}{b} C \ldots\ldots\ldots\ldots (A \text{ fixed}) \ldots\ldots\ldots (25)$$

When the load on AB is unsymmetrical,

$$M_{AB} = -C_{AB} - \frac{2}{b} C_{BA} \ldots\ldots\ldots (A \text{ fixed}) \ldots\ldots\ldots (26)$$

$$M_{BA} = -\frac{b-4}{b} C_{BA} \ldots\ldots\ldots\ldots (A \text{ fixed}) \ldots\ldots\ldots (27)$$

APPLICATION OF UNIT FRAME TO FRAME ANALYSIS

Figs. 1, 2 and 8 show unit frames with but one span loaded, to which formulas given herein will apply. Large frames of the kind

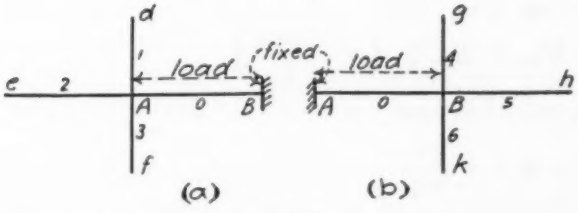


FIG. 6

shown in Fig. 7 may be analyzed approximately by the application of unit frames. Consider Fig. 7(a) for example. By considering the span to the right of joint A as the loaded span, the first unit frame would be that shown in Fig. 8(a). The columns at joint B and the girder at the right of B would be so affected by the loads on the floors

above, below and beyond B, that the rotations at the joints D, C, and E could be approximately taken as equal to Θ_B, but of opposite direction (hence opposite in sign). Then, from eq. (4), the value of n for these members meeting at B would be 2.

At the joint A there would be no rotational effect from loads. Hence this joint could be assumed fixed. The moment M_{AB} is next determined in the unit frame.

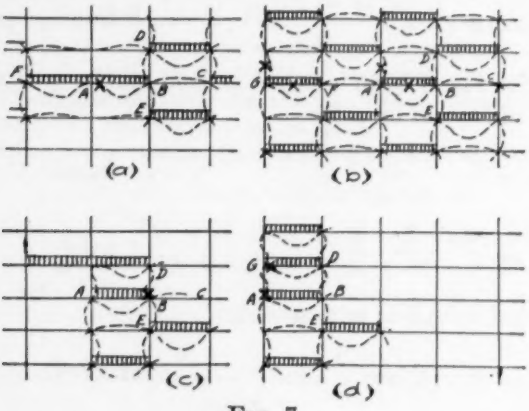


FIG. 7

The unit frames in Fig. 8 show their application to the frames loaded as shown in corresponding parts of Fig. 7. The n values are also indicated for the frame conditions shown. The choice of these n values is based upon an estimate of the frame distortion as shown by the dotted lines in Fig. 7. It is of interest to note that

$$M_{BD} = 2\,EK\,(2\Theta_A + \Theta_D)$$

may be written in the form

$$M_{AB} = EK\,\Theta_A\left(4 + \frac{2\,\Theta_D}{\Theta_A}\right)$$

from which it is clear that

$$n_D = 4 + \frac{2\,\Theta_D}{\Theta_A} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

From this expression it is seen that the value of n is dependent on the ratio of the rotations at the two ends of AD; or upon Θ_D/Θ_A. This ratio may usually be estimated even though the rotations themselves cannot. On such basis the values of n in Fig. 8 were chosen, after inspection of Fig. 7.

POSITION OF LIVE LOAD FOR MAXIMUM AND MINIMUM MOMENTS

The maximum or minimum moments are determined by considering arrangements of live loads to produce the greatest or least restraints

at adjacent joints. The arrangements in Fig. 7 are for uniform spans and story heights, to produce maximum moment at the point marked X. For example, in Fig 7(a) the joint B is made to rotate as much as possible while the joint A is held without rotation. In 7(b), joints A and B are both rotated as much as possible.

Fig. 8 gives the unit frames for computing the values for loadings shown in Fig. 7. (8(a) corresponds to 7(a), and so on). It also indicates the values of n to be used, in accordance with eqs. (2) to (5).

BEAMS WITH HAUNCHES AND CURVED SOFFITS

It is frequently desirable to put brackets or haunches at the junction of beams and columns; or to curve the soffit (bottom face) of a beam or girder. Such change in the section of the member affects

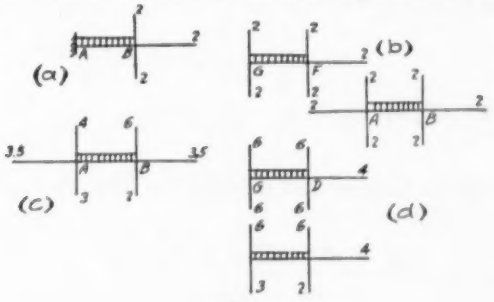


FIG. 8

the moment distribution. A simple modification of the foregoing method permits its application to such members of frames. Eqs. (1) may be written in the general form:

$$M_{AB} = EK_c\,(C_1\Theta_A + C_2\Theta_B) - C'_{AB} \quad \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

$$M_{BA} = EK_c\,(C_2\Theta_A + C_1\Theta_B) + C'_{BA} \quad \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

in which $K_c = I_c/l$ and I_c = moment of inertia at the center of a haunched member; or at the small end of a tapered member.

C_1 , C_2 = coefficients dependent upon form of haunch or soffit of member.

C'_{AB} , C'_{BA} = Load terms equal respectively to the end moments at A and B of a *fixed beam of the same form as AB and carrying the same load.* They will be equal if the member AB is symmetrically formed and loaded.

Consider now a restraining member, as AX. $M_{XA} = O$, that is, if the member is hinged or simple supported at X,

$$M_{AX} = \left[\frac{C_1^2 - C_2^2}{C_1}\right] EK'_c\,\Theta_A \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

when K'_c is the value of I_c/l for the member.

If $\Theta_X = O$, that is, the member is fixed at X,

$$M_{AX} = [\,C_1\,]\,EK'_c\,\Theta_A \quad \ldots\ldots (32)$$

If $\Theta_X = -\Theta_A$,

$$M_{AX} = [\,C_1 - C_2\,]\,E\,K'_c\,\Theta_A \quad \ldots\ldots (33)$$

If $\Theta_X = +\Theta_A$

$$M_{AX} = [\,C_2 + C_1\,]\,E\,K'_c\,\Theta_A \quad \ldots\ldots (34)$$

Eqs. (31) to (34) correspond to eqs. (2) to (5) above; and in general, it is seen that the term in [ ] is the value of n. In these four equations C_1 and C_2 refer to each restraining member in turn as its n-value is determined.

Let $\sum_{1}^{3} nK'_c/K_c + C_1 = a$

and $\sum_{4}^{6} nK'_c/K_c + C_1 = b$

Then proceeding as before and solving for the end moments, there results:

$$M_{AB} = \frac{-(a - C_1)\,(bC_{AB} + C_2C_{BA})}{ab - C_{22}} \quad \ldots\ldots (35)$$

$$M_{BA} = \frac{(b - C_1)\,(C_2C_{AB} + aC_{BA})}{ab - C_{22}} \quad \ldots\ldots (36)$$

in which C_1, C_2, C_{AB} and C_{BA} refer only to the loaded member AB.

It is interesting to note that by making $C_1 = 4$ and $C_2 = 2$ in eqs. (31) to (36) one at once arrives at the terms for a frame having members of constant section for their full lengths, as given earlier.

The values of C_1, C_2, C_{AB} have been computed by Evans and are given in diagram form.* These curves are intended for moving loads and are suitable for preparing influence lines of moments.

Example 3. Suppose the moments at A and B of AB in the frame of Fig. 9 to be desired. Then from eq. (31) and Diagrams 13 and 17,

$$n_{AD} = \frac{10.8^3 - 4.7^3}{10.8} = 8.73 \text{ and } n_{AD}K'_c/K_c = \frac{(8.73 \times 2)}{1} = 17.42$$

For the column AE, eq. (3)

$$n_{AE}\,K/K_c = 3.$$

whence $\sum_{E}^{D} nK/K_c = 17.42 + 3 = 20.42.$

*See Evans, L. T., Modified Slope-Deflection Equations, JOURNAL AM. CONCRETE INST., Oct. 1931, *Proceedings* Vol. 28, p. 109. Mr. Evans' 17 diagrams are here republished—pages 161 to 169. EDITOR.

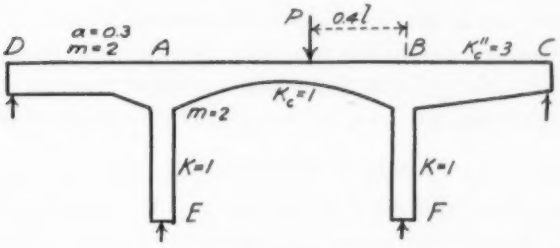


FIG. 9

Taking C_1 for AB from Diagram 1 as 22.5,

$$a = 20.42 + 22.5 = 42.92.$$

Similarly, from Diagram 9,

$$n_{BC} = \frac{32.5^2 - 8.2^2}{32.5} = 29.9$$

and $$n_{BC}\, K''_c/K_c = \frac{(39.9 \times 3)}{1} = 119.7$$

Since the value of $n_{BF} = 3$, and $n_{BF}\, K/K_c = 3$

$$\sum_{C}^{F} nK/K_c = 119.7 + 3 = 122.7$$

Taking C_2 for AB from Diagram 1 as 17.5,

$$b = 122.7 + 17.5 = 140.2$$

The load terms C_{AD} and C_{BA} are taken from Diagram 1 and are respectively 0.115 Pl and 0.223 Pl, the latter being the same as C_{AB} for $k = 0.6$ since the beam is symmetrical in form. The moments at A and B thus become

$$M_{AB} = \frac{(22.5 - 42.9)\,[\,(140.2)\,(0.115) + (17.5)\,(0.223)\,]\,Pl}{(42.9)\,(140.2) - (17.5)^2} = -0.072\ Pl$$

$$M_{BA} = \frac{(140.2 - 22.5)\,[\,(17.5)\,(0.115) + (42.9)\,(0.223)\,]\,Pl}{(42.9)\,(140.2) - (17.5)^2} = +0.244\ Pl$$

DETERMINATION OF MOMENTS OF INERTIA AND K

Since the moment distribution in a frame is dependent only upon the *relative* stiffness among the members, any simple but consistent procedure in determining K for each member will suffice. In general, for beam or column sections, the gross concrete area may be used, and the steel area is transferred to equivalent concrete area by multiplying by E_s/E_c. Then

$$I = I_{concrete} + \frac{E_s\, I_{steel}}{E_c} \qquad (37)$$

all taken about the centroid of the gross concrete areas.

In flat slabs, use a width equal to the column strips.

In joist or beam construction, use a group of joists or beams included in a width of one-half the transverse column spacing, but not to exceed a width equal to one-half the span of the joists or beams.

VALIDITY OF RESULTS

The method here presented is subject to the following limitations:

(a) Those limitations which are inherent in the slope-deflection method of analysis; for example, rigid connections are assumed, and no change in length of members is considered.

(b) As presented, the method does not provide for the moments in frames arising from side-drift produced by unsymmetrical framing or loading.

(c) As presented, the method calls for the arbitrary evaluation of the restraint of the far ends of the various restraining members of the so-called "unit frame"; and proposes a basis for reasonably close evaluation of such restraint factors.

There is no method of elastic analysis sufficiently compact and practical of application that does not rest fundamentally upon the same assumptions and limitations of the slope-deflection method. In fact, it is generally accepted as one of the so-called "exact" methods of analysis, and as such, is recognized to represent the practical ultimate of accuracy.

The error introduced due to neglect of side-drift produced by unsymmetrical load or span arrangements is usually small. In markedly unsymmetrical frames, however, it should be taken into account by "exact" analysis. It is worth noting that to date common practice neglects such side-drift effect, except at the hands of those designers possessing the analytical equipment to make the necessary refinements.

The errors introduced by inaccuracy of selection of restraint factors for terminals of restraining members may be considered as the more important of those noted above. These errors depend on the degree to which the designer becomes adept at selection of the factors. Usually it is the case that the factors can vary only between the well-established values set forth herein, and when this is true, the errors are sufficiently small to be admissible. The designer will find that the more unusual cases will yield to ready recognition and interpretation of special conditions of restraint.

It appears reasonable to the writer that while it is laudable to attempt to achieve as nearly accurate an analysis as practicable, nevertheless the ultimate degree of accuracy of any analysis need not

be in excess of that degree to which the loadings involved, the properties of the materials actually incorporated in the final structure, and the precise distribution of stresses on cross-sections may be accurately known to the designer.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by February 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for March-April, 1937.*

The following Diagrams, 1—17, are from L. T. Evans' paper "The Modified Slope Deflection Equations", JOURNAL AMERICAN CONCRETE INSTITUTE, October 1931, *Proceedings*, Vol. 28, p. 109.

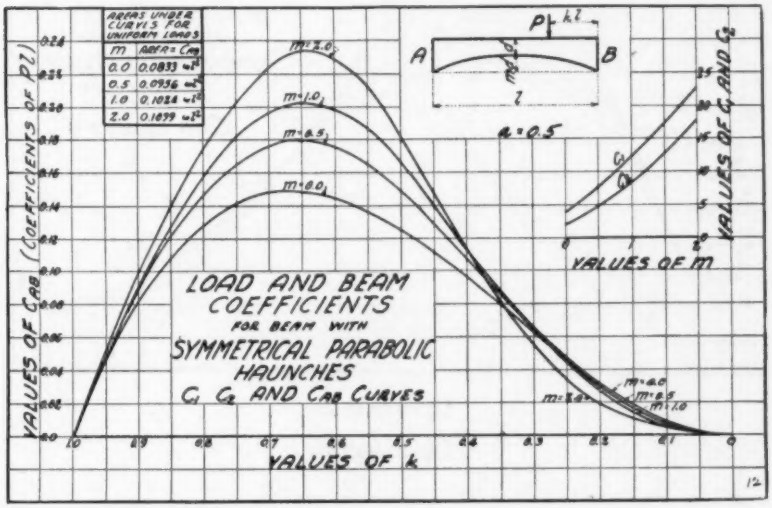
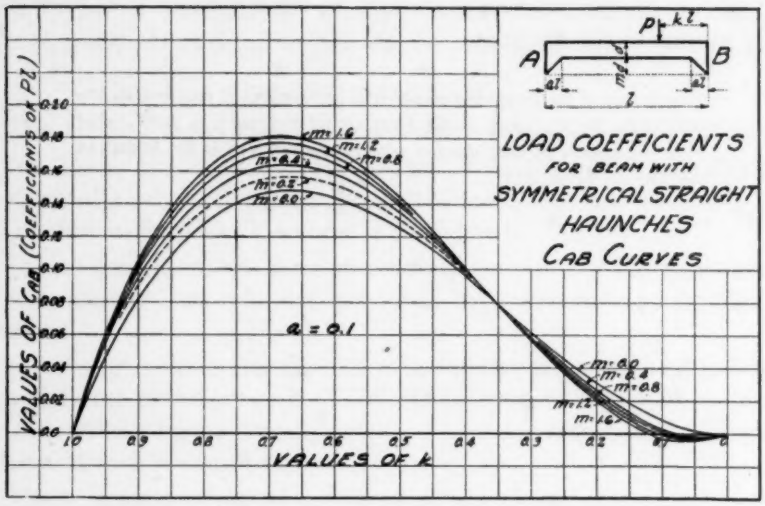


DIAGRAM 1



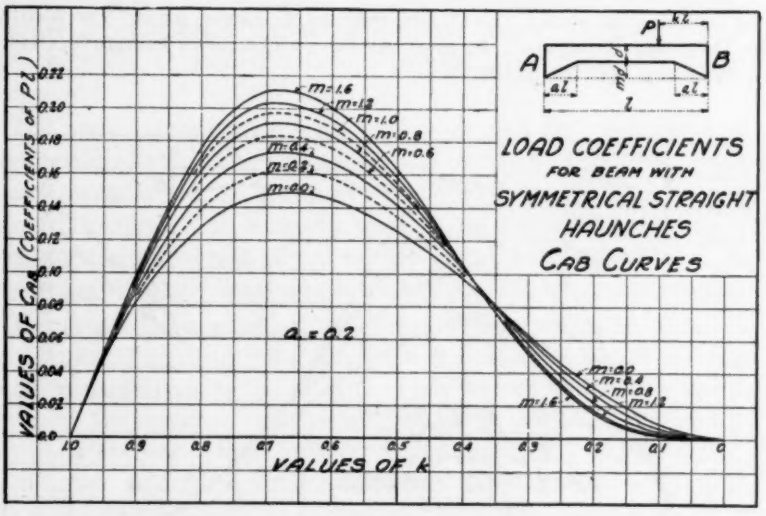


DIAGRAM 2

DIAGRAM 3

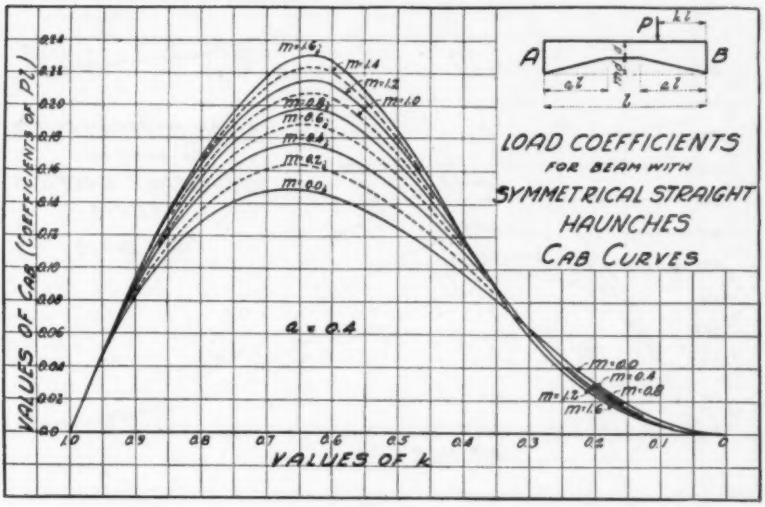


DIAGRAM 4

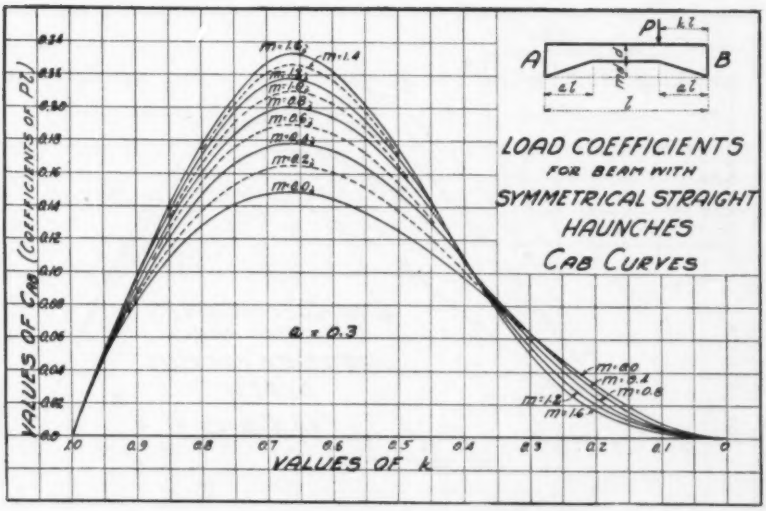


DIAGRAM 5

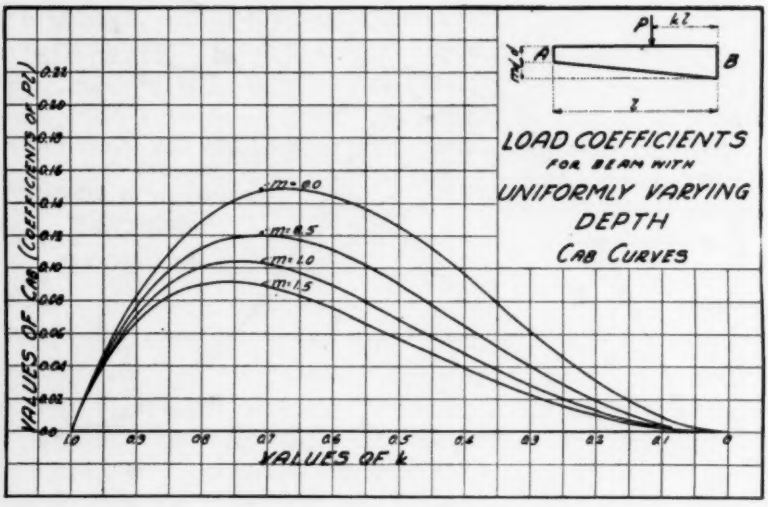


DIAGRAM 6

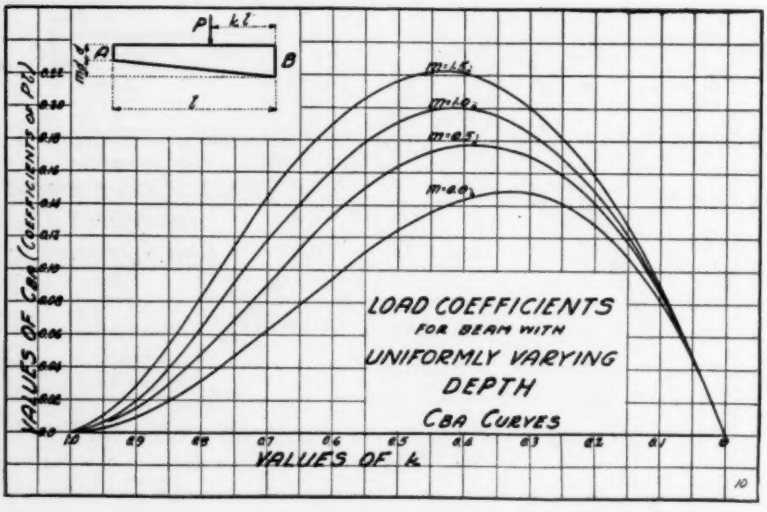


DIAGRAM 7

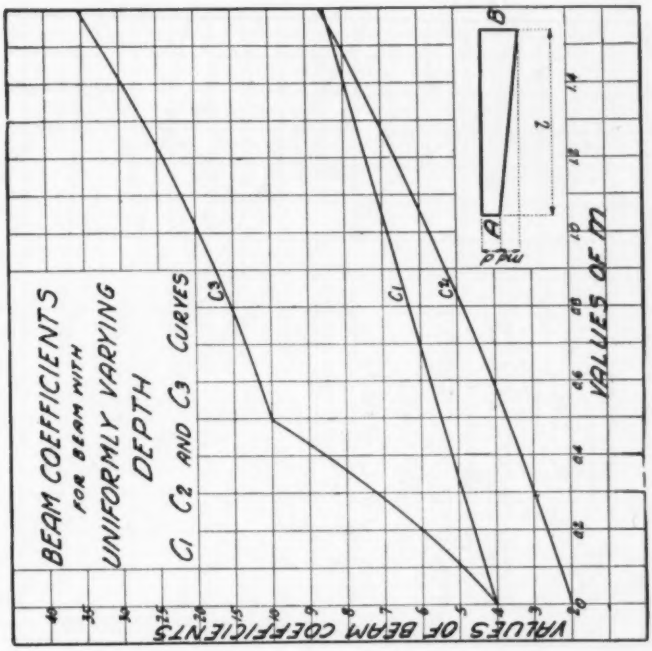


DIAGRAM 9

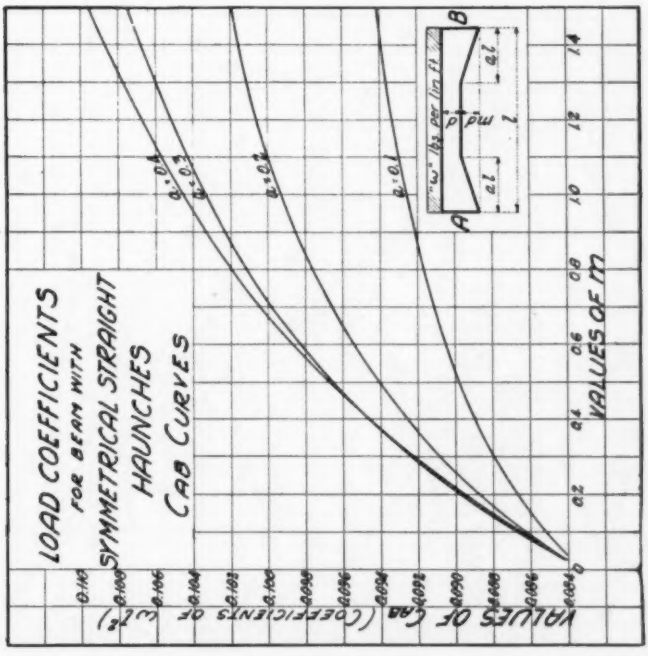


DIAGRAM 8

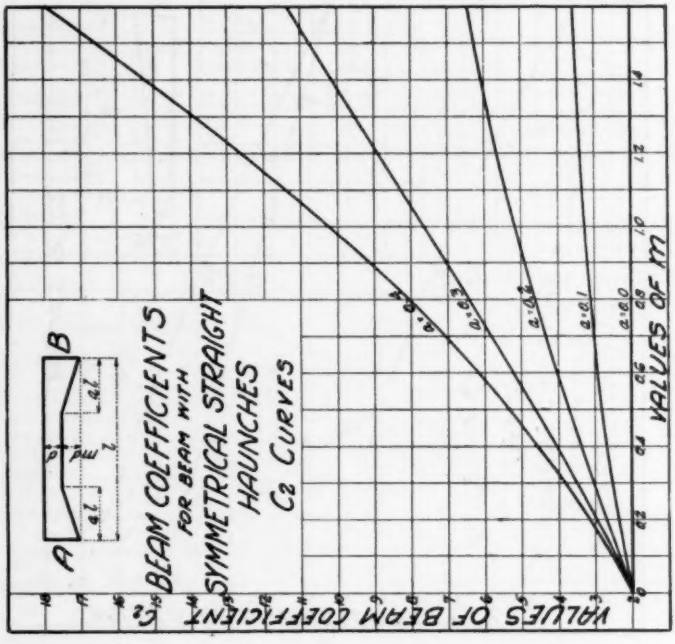


DIAGRAM 11

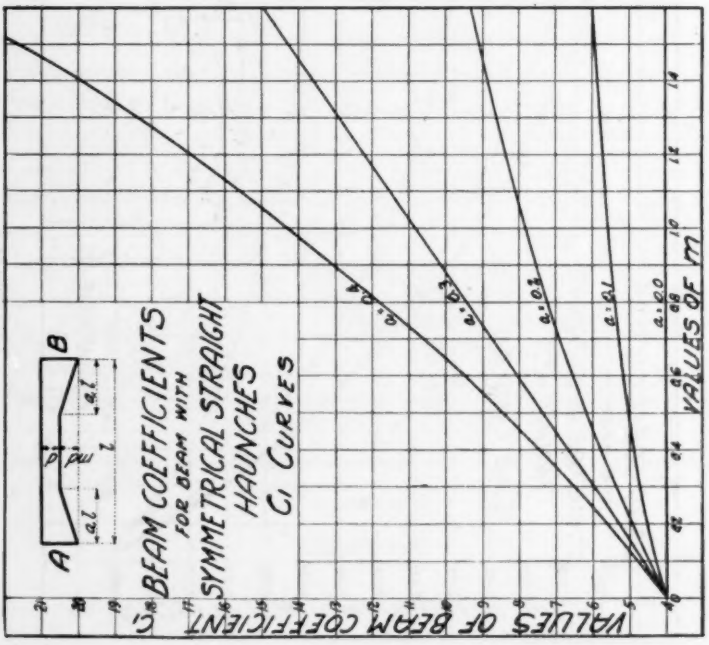


DIAGRAM 10

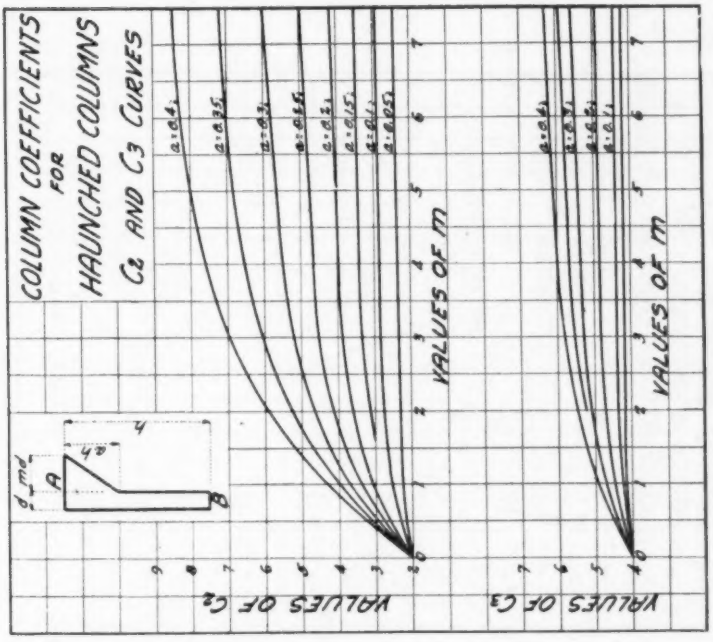


DIAGRAM 13

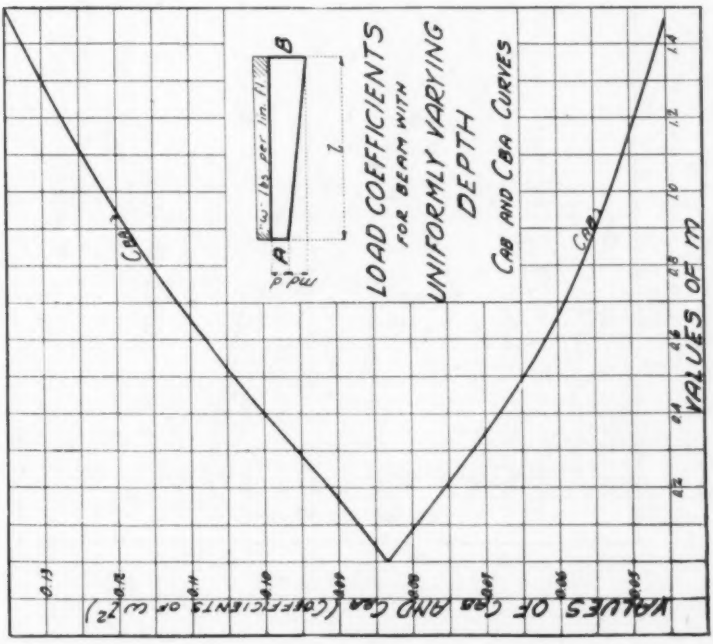


DIAGRAM 12

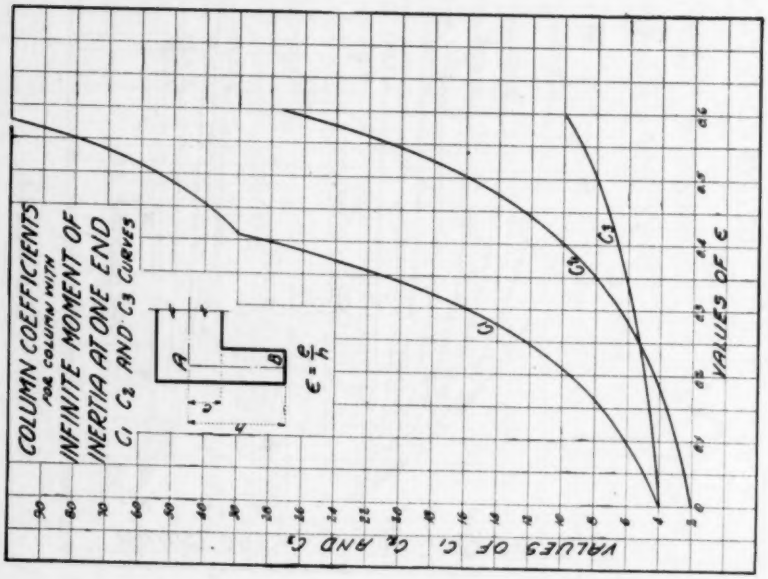


DIAGRAM 15

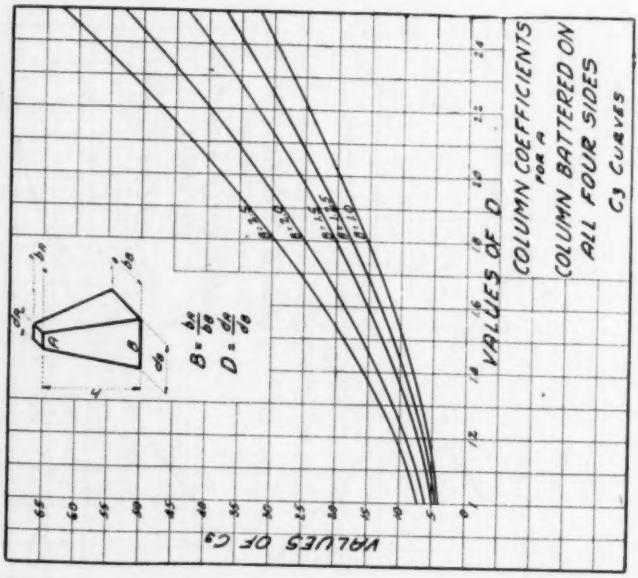


DIAGRAM 14

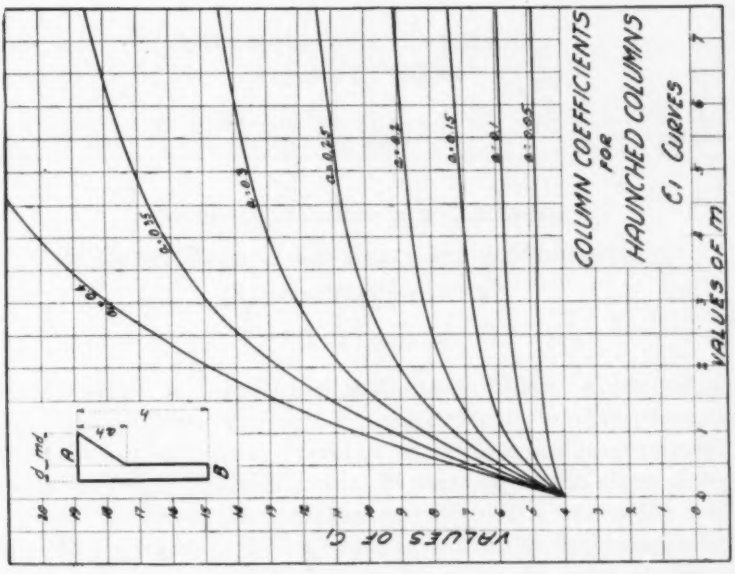


DIAGRAM 17

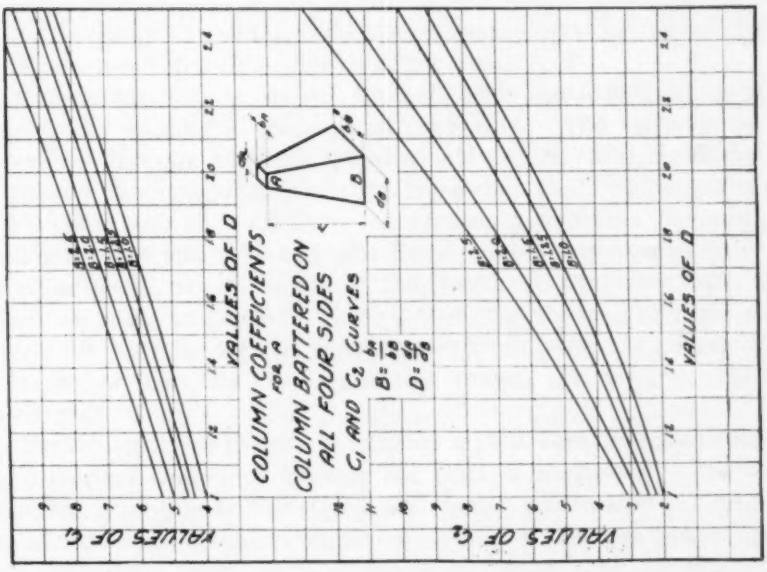


DIAGRAM 16

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . MARCH-APRIL 1937

Discussion of a paper by S. C. Hollister:

A Short Method for Computing Moments in Continuous Frames*

A. FLORIS†

The simplicity and directness of the slope deflection method is well known. Its application to rigid frames, however, involves the solution of simultaneous equations, a tedious and time-consuming process.

There are other methods available based upon the slope deflection equations. In these the simultaneous equations are either avoided or solved by iteration. In both cases the solution is accomplished by a step by step elimination of the unknowns. In spite of their simplicity these methods are laborious, requiring considerable time. For this reason any method which shortens the necessary computations will be appreciated by engineers in practice.

Continuous frames under vertical loads are analyzed approximately by loading a single panel each time. The influence of the members framing into this panel is only taken into consideration. Precisely the same idea, although somewhat modified by the use of the undetermined coefficients of restraint, underlies the author's analysis. By means of the unit frame loaded symmetrically or unsymmetrically, the moments at the joints are evaluated with sufficient accuracy and speed. Nevertheless, the guessing of these coefficients of restraint by the unexperienced computor, as suggested by the author, can lead to appreciable errors. It requires skill and judgment.

Regarding the side-drift it is often stated that in reasonable unsymmetrical frames or loading the drift is negligible. The writer believes that this is overstated and based presumably on intuition rather than on actual calculations. Some of the cases investigated by the author might have, under certain conditions, an appreciable side-drift.

*JOURNAL, Amer. Concrete Inst., Nov.-Dec. 1936, p. 147.
†Los Angeles, Calif.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . NOVEMBER-DECEMBER 1936

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers.

Solved and unsolved problems of clinker research

HANS KUHL, *Tonindustrie Zeitung*, Vol. 60, No. 48 and 49, 591-3, 607-9, June 15 and 18, 1936.
Reviewed by A. E. BEITLICH

The author describes the reactions during the burning and cooling of portland cement clinker on hand of the three-component diagram CaO — Al_2O_3 — SiO_2 and the development of the various formulas for the calculation of the degree of lime saturation, the highest possible amount of calcium oxide that can be carried without running danger of overliming the raw mixture.

The specific surfaces of cements and powdered rocks

H. ELSNER VON GRONOW, *Tonindustrie Zeitung*, Vol. 60, No. 52, 643-6, June 29, 1936.
Reviewed by A. E. BEITLICH

It is shown how distribution curves can be plotted for the grain sizes of cements and powdered rocks, obtained by the sedimentation methods of Andreasen and Haegermann, Kuhl and Czernin which are commonly used in Germany. The specific surfaces can be determined easily from these diagrams without further complicated calculations. The possibilities for eliminating doubtful points in the calculation of surfaces of rock powders used as fillers for bituminous road construction are discussed.

Analysis of continuous frames by balancing angle changes

L. E. GRINTER, *Am. Soc. C. E. Proc.*, Vol. 62, No. 7, Sept. 1936, p. 995-1011. Reviewed by H. J. GILKEY

Procedure parallel to that of Hardy Cross for balancing fixed-end moments. Advantages claimed are that the method is practically self-checking and that at completion of analysis there are at hand the necessary data for sketching the deflected structure. No claim is made that the method is either shorter or simpler than the Cross method except indirectly because of the more ready availability of slope changes. The author recommends the method for the use of those accustomed to thinking in terms of joint rotations and deflections.

Sulfides in basic slags and slag cements

VIERI SEVIERI, *Tonindustrie Zeitung*, Vol. 60, No. 56 and 57, 691-3, 703-4, July 13 and 16, 1936.
Reviewed by A. E. BEITLICH

Several investigators have demonstrated that the calcium sulfate of basic blast furnace slags can not be oxidized to sulfate in the presence of air, water or sea water. Even if the sulfides in slag cements could be oxidized, only calcium sulfate and not free sulfuric acid would be formed. The calcium sulfate, imbedded in a basic medium

could not attack steel reinforcements. The presence of sulfides in slags or slag cements is therefore not harmful for reinforcements; they even improve the hydraulic properties of the cements.

The effect of sodium chloride on cement

W. WITTEKINDT. *Tonindustrie Zeitung*, Vol. 60, No. 65, 797-8, Aug. 13, 1936.
Reviewed by A. E. BEITLICH

Two grams of portland cement, agitated for 24 hours with 200 cc. of water, yielded 0.2060 g. of calcium oxide in the filtrate. With a 0.1 normal solution of sodium chloride 0.2064 g. were dissolved and with a 0.1 normal solution of potassium chloride 0.2088 g. were dissolved. With increase of the concentration of the salt solution and longer agitation periods, more calcium oxide was dissolved. This was also true for hydrated portland cement; the effect of the salt was less pronounced with progressing age of the cement.

The determination of the proportions of constituents in solid masses (concrete)

R. GRENGG and A. SCHMÖLZER. *Tonindustrie Zeitung*, Vol. 60, No. 54, 665-8, July 6, 1936.
Reviewed by A. E. BEITLICH

The authors illustrate several methods which appear promising for the determination of the proportions of cement in hardened concrete. The principle of these methods is that certain admixtures are mixed in with the cement which do not react with the cement, do not dissolve and do not affect the properties of the concrete. As possible admixtures are listed: barium sulfate, graphite, ground glass, etc. At later ages these admixtures can easily be detected by chemical means or under the microscope and from the amount present, the amount of cement can be calculated.

Interaction between rib and superstructure in concrete arch bridges

NATHAN M. NEWMARK, *Am. Soc. C. E. Proc.*, Vol. 62, No. 7, Sept. 1936, p. 1043-1061.
Reviewed by H. J. GILKEY

Concludes that net effect of interaction is normally small and that the ordinary method of designing arch structures by neglecting interaction is usually sound. Sometimes possible to obtain greater economy by detailing floor and columns to reduce their rigidity rather than attempting to stiffen them with a view to aiding the rib. In the latter, there is always the possibility of a crack in the deck destroying part of the interaction. Also stresses due to temperature and movements of foundations are high in the integral type of structure.

Some viewpoints regarding sound insulation

EMRIK LINDMAN, *Betong*, No. 3, 1936, pp. 97-115. Reviewed by INGE LYSE

The problem of sound insulation has probably been taken more seriously in Europe than here. This particular paper points out that even in Europe the houses were generally better insulated before the war period than in the decade immediately following. Since 1930 the problem has again come to the front and considerable attention is now given to it. The author discusses the theory of sound insulation and classifies the insulating quality into seven groups, ranging from excellent to extremely poor. The decibel method of sound classification is used. The author presents an analytical study of the problem and shows graphically the relationships obtained. Different types of concrete and reinforced concrete construction are represented and their relative merits for sound insulation are shown. The author points out that only approximate values are as yet available on sound insulating qualities and that future experimentation may warrant corrections in the values given in the paper.

New road surfacing in reinforced concrete

C. J. H. VELLE. *Schweizer, Zeit. Strassenwesen*, 1936, 22 (3) 46-8. Abstract No. 227 appears in Road Abstracts, June, 1936.

The concrete wearing course is laid over a series of slabs made from a harsh mix premolded under pressure. The slabs are placed on the subgrade in such a manner as to produce longitudinal and transverse troughs about 5 cm. (2 in.) wide. The bottoms of the troughs thus formed are covered with special paper. Reinforcement is arranged in the troughs and upon the slabs and the wearing course concrete is then placed. The edges of the road are formed by reinforced beams cast integrally with the wearing course and carried to a depth of about twice the thickness of the complete road crust. The transverse reinforcement of the road is anchored in these beams. It is claimed that a road constructed in this manner can be accurately designed to carry the loads to which it will be subjected. Economy is obtained by the use of factory-made slabs for which cheap aggregates may be employed. The reinforcement can be laid accurately in place. The plane under-surface of the road facilitates expansion and contraction and reduces cracking. Fewer joints are needed. Some notes are included on design calculations for this type of surfacing.

Failure and safety factor in reinforced concrete beams

RUDOLF SALIGER, *Beton und Eisen*, Vol. 35, No. 19 and 20, Oct. 5 and 20, 1936.
Reviewed by INGE LYSE

Professor Saliger points out that although the agreement between calculated and observed stresses at relatively low loads such as used in ordinary design is quite satisfactory, the stress distribution at failure is entirely different. Experiments show that for low percentages of steel the calculated stresses in the steel at time of failure are considerably greater than the yield-point stress, while for high percentage of steel the calculated stress agrees fairly well with the yield-point stress. Furthermore, the resistance of the concrete has been found to be greater than calculated when high strength steel or high percentage of steel has been used. The author therefore undertakes to study the condition of the reinforced concrete beam at time of failure. This study leads to the conclusion that while the common reinforced concrete theory with n values of 10 or 15 gives satisfactory stress values within working loads, it is entirely erroneous at time of failure. The position of the neutral axis is generally different than that given by the ordinary reinforced concrete design methods and the stress in the concrete loses its significance as far as factor of safety is concerned. Professor Saliger shows how these different conditions may be taken care of in design and how the present methods of design should be altered in order that the actual factor of safety in reinforced concrete structures be accepted as the criterion of design.

The value "n"

FRITZ EMPERGER, *Beton und Eisen*, Vol 35, No. 19, Oct. 5, 1936, p. 324-332. Reviewed by INGE LYSE

Lately there has been a considerable discussion in European engineering magazines of the n value in reinforced concrete design. Mr. Emperger presents his views, discussing the different values used in different countries; reviews some of the experimental work which has been done in this field and sums up his studies with these conclusions:

1. For the design of low percentage of reinforcement in beams the neutral axis may be taken at midheight, that is $x = \frac{1}{2} h$, so that the resisting moment of the beam becomes: $M = 5/6\ h.f.f_c'\ A_s$. If, however, the common design formulas are desired maintained at least a standard value for n should be adopted universally so that the present variations can be eliminated.

2. When the percentage of reinforcement is so high that the calculated steel stresses will reach yield-point of the steel at the same time as the compressive strength of the concrete is reached, a constant value n should be used. This value should be determined on the basis of a trapezoidal stress distribution in the concrete at time of failure. The moment of resistance of the beam will then become:

$$M = 5/6\ b.x.\ (h - 19/45\ x)\ .\ fc'$$

$$\text{where: } x = \frac{9n.p.}{5}\left(-1 + \sqrt{1 + \frac{2.5}{9N.p.}}\right)$$

Mr. Emperger's paper is worthy of serious consideration by our building code committees.

Problems of road research

R. E. STRADLING. Peter le Neve Foster lectures. Royal Society of Arts, London. *Roads and Road Construction*, Aug. 1, 1936 HIGHWAY RESEARCH ABSTRACTS

Regarding improvements in portland cement concrete quality, one of the most serious problems is the variation from day to day, even from mix to mix, of the water-cement ratio. It has been shown by W. H. Glanville and F. N. Sparkes, however, that subjection of a mass of saturated sand or coarser aggregate to vibration results in reducing the water content to a certain definite amount. Based upon this premise, a vibrator has been developed which consists essentially of a container, the bottom of which is perforated to allow water to drain away. The material, placed in this and saturated with water, is then vibrated by a special arrangement of an out-of-balance mass rotated by a motor. After laboratory trials the apparatus was used on actual road work, and now appears to be a thoroughly practical device. The container can be used to measure the sand and aggregate quantities, in addition to its use in bringing them to constant water content. The success of the apparatus may be judged from the following figures of results obtained on a road construction project. Neither mixing time nor labor costs were appreciably increased, while decidedly greater uniformity was achieved.

EFFECT OF USE OF VIBRATOR APPARATUS

	No Vibration	Sand Vibrated	Sand and Gravel Vibrated
Water Content (Variation from Mean)	±17%	± 5%	± 2%
Crushing Strength	6200—2300 p.s.i.	6700—4500 p.s.i.	6800—4500 p.s.i.
(Variation from Mean)	±50%	±18%	±18%

Construction of the bridge of Bonnant over the Loire

M. VALENTON (Engineer of Ponts et Chaussees) *Les Annales des Ponts et Chaussees*, Aug. 1935. Reviewed by B. MOREELL

Before deciding upon the type of highway bridge to be constructed over the Loire near Bonnant, the Ministry of Public works had comparative studies made of a suspension bridge, a continuous steel girder bridge, and a reinforced concrete multiple-arch bridge. In the interests of both economy and aesthetics, it was decided to construct the multiple-arch concrete bridge. The bridge has a total length of 205 meters between abutments and consists of five arches of 38 meters span each.

The arches are the two-hinged barrel type, 6 meters wide. The hinges are the Mesnager type. The roadway is supported on thin transverse walls, 2.59 m. c. to c. The width of roadway is 5.50 m. and there are two sidewalks one meter wide each, the sidewalks being cantilevered out from the roadway slab. There is an expansion joint in the superstructure at each pier.

An interesting feature of this bridge is that the rise of the arches increases from the shore to the center of the bridge. The spans next to the shore have a rise of 3.80m. The intermediate arch has a rise of 4.0075 m. and the central arch a rise of 4.1925 m. While this procedure undoubtedly increases the cost of the form work, it results in a very pleasing appearance.

The vertical walls supporting the deck are articulated at the base where they join the arch and at the top where they are connected to the deck. These are also Mesnager articulations.

The piers are founded on clay. Considerable data are given relative to the method of installing the piers and the preliminary investigations.

Method for determining the quality of clinker

WILHELM ANSELM, *Zement*, Vol. 25, No. 38, p. 633-644, Sept. 17, 1936.

Reviewed by L. T. BROWNMILLER

By quality of clinker the author refers to the degree of burning of the clinker; that is according to his general classification clinker may be underburned, well-burned or overburned. His method for evaluating quality is by a determination of the liter weight of a certain screened fraction of the clinker. The liter weight increases gradually from underburned to overburned clinker. The method is proposed as a control during the burning process. However no definite limiting values of the liter weight can be set for well-burned clinker because those values will depend on the composition of the raw mix and type of kiln. It is therefore necessary to predetermine the liter weight of well-burned clinker for each raw mix, and use that value as the control for that particular mixture only. But the values still may not represent accurately the degree of burning if there are slight variations in composition of the raw mix, particularly in those constituents such as Al_2O_3 and Fe_2O_3 on which the amount of liquid largely depends.

An extensive correlation was made by the author between liter weight of clinker and the properties of the cement,—soundness, shrinkage, strength, free lime, grindability. A microscopic study was also made of clinker of various degrees of burning which showed that $3CaO.SiO_2$ increased with increasing liter weight; $2CaO.SiO_2$ generally remained constant, while the ground mass decreased in amount with increasing liter weight.

The article is one of the few published reports on the test and the correlation of data is more comprehensive here than in other articles on the subject.

Concrete pavement in swamps

Concrete Highways, July-August, 1936. HIGHWAY RESEARCH ABSTRACTS

The North Carolina State Highway Department has demonstrated that when a subgrade is unable to support the weight of pavement under traffic, the beam strength of concrete can still be utilized to advantage. In the low, flat tidewater section of eastern North Carolina, the soil near streams and bays is sometimes so marshy that roads built upon it sink from sight. After thorough study it was decided that the cheapest type of construction in these circumstances was a reinforced concrete slab resting on piling, serving both as a bridge and pavement. One instance of the success of this method concerns an original pavement built in this region on a log mat covered with gravel, which had gradually settled down into the muck from the weight and vibration of traffic. As the slab continued to sink, maintenance material was placed over the roadway until there was a fill of 3 to 4 feet. Short pilings were then tried as supports for the slab, but these failed. Borings made to determine the nature of the foundation showed the following soil profile: peat, 24 ft.; quicksand,

13 ft.; soft marl, 3 ft.; quicksand, 10 ft.; and soft blue marl, 11 ft. To meet this condition long pilings were driven to firm bearing, and the subgrade brought to proper level. Cone-shaped holes were then made around the piling with a metal form, and reinforcement put into position. A piling cap was then placed monolithic with the slab, and concrete for a whole panel was placed in a continuous operation, and mixed, spread, and finished as for an ordinary concrete pavement. The unusual features of the design are the beam, monolithic with the pavement, that extends along the center joint and transverse joints, and the conical cap for the piling. On the outside was placed an integral curb. The total cost of the 1,128 ft. of supported 8-in. 18-ft. slab was $17,630, or $7.81 per sq. yd. of pavement. The entire project was completed in 67 working days, and was so successful that a second section was undertaken.

Effect of the composition of the concrete on its workability and strength

F. V. EMPERGER and K. KUGI, Austrian Committee for Reinforced Concrete, Vienna, 1936.
Reviewed by INGE LYSE

This excellent bulletin consists of six separate reports of investigations at different institutions and a summary study by Emperger and Kugi. The first report deals with strength studies of concrete at the Technical Institute at Vienna and reported by Rinagl and Müller. The water-cement ratio theory was the principal object of the study and the results showed the general relationship between this ratio and the strength of the concrete. However, different cements and aggregates gave widely different results for any given water content.

Studies at Gras were of the workability index as measured by Power's remolding effort and the strength results as compared with Abram's water-cement-ratio law.

At the Vienna City Testing Laboratory, strength results were compared with the formulas by Abrams, Bolomey, Feret and Graf. The three types of cements used showed considerable difference in strengths.

Workability and strength were the important factors studied. Tests made at the testing laboratory of the Material and Technological Museum. The results are presented in tables and diagrams and show a fair degree of uniformity.

The fifth report is also a study of the relationship between strength and water-cement ratio. The results showed that the water-cement ratio curve is not the same for different materials and different conditions, but that a number of variables influenced the final strength. O. Stern presents an analysis of concrete composition by mathematical means. The significance of this study is difficult to evaluate.

Emperger and Kugi give an excellent summary of these reports, pointing out the effect of the various constituents in concrete and how the trends of the results from the different institutions indicate definite conclusions. The bulletin contains 140 pages with numerous tables and figures.

Report of the 59th meeting of the Association of German Cement Manufacturers

Published by Zementverlog, Berlin, 1936. Reviewed by INGE LYSE

This bulletin gives reports of nine papers presented at meetings in Berlin, March 31 and April 1, 1936, and the subsequent discussions: "Drawbacks in Our Cement Specifications," Walter Dyckerhoff; "Selecting Cement for Highway Construction," Otto Graf; "The Chemical Composition of Portland Cement," G. Haegermann; "Developing Cements for Highway Construction," H. E. Schwiete; "Report from the Association Research Laboratory," G. Haegermann; "Report from Governmental

Research Laboratory," Lothar Krüger; "The Properties of Cements with High Magnesium Content," W. Eitel; "Lime Silicates and Lime Aluminates in Water Solutions," R. Nacken; "Solved and Unsolved Problems in Clinker Research," Hans Kühl.

Some of the outstanding features of these papers should be mentioned. Mr. Dyckerhoff urged modification of the standard tests of cements so that the test results might be more indicative of the real quality of the cement. He advocated the use of graded sand for mortar tests with flexural as well as compressive strength studied on plastic mixes.

Professor Graf pointed out the importance of the compressive and flexural strengths and their relation in highway construction. He discussed the relation between tensile and flexural strengths, the volume changes, the fineness of the cement, the chemical composition and the uniformity in behavior. Field observations were discussed and further studies outlined.

Mr. Haegermann and Mr. Schwiete emphasized the importance of flexural strength and low shrinkage for cement for highway construction and discussed means of obtaining such results by proper composition of the cement.

The reports from the testing laboratories dealt with the shrinkage and cracking problem, with securing representative cores from pavements, the effect of hot cement and the resistance of concrete to aggressive water.

The last three papers dealt primarily with the chemical ingredients of one cement. Each paper was discussed at length and the report becomes a valuable addition to our literature.

Calcium ferrite hydrates

HELLMUTH HOFFMANN, Vol. 25, Nos. 8, 9, 39, 40, 41, p. 113-117, 130-132, 675-680, 693-698, 711-716, Feb. 20 and 27, Sept. 24, Oct. 1 and 8, 1936. Reviewed by L. T. BROWNMILLER

The experiments reported in this paper relate to the products of hydration of the iron compounds which may occur in portland cement clinker, namely, $2CaO.Fe_2O_3$, $4CaO.Al_2O_3.Fe_2O_3$ and $CaO.Fe_2O_3$. No experimental evidence is offered to show that the products of hydration of these pure compounds are the same as the hydrates of the iron compounds of cement. Any interpretations of the results of the experiments in terms of portland cement are therefore based on the assumption that the compounds used are identical with those present in cement; and that their hydration in cement proceeds in the same way as under the experimental conditions outlined.

Calcium ferrite hydrates were synthesized first of all from calcium hydroxide and ferric hydroxide. Two hydrates, which were white in color, were formed: $4CaO.Fe_2O_3$, $14H_2O$ and $4CaO.Fe_2O_3.7H_2O$. This is in agreement with the work of Pelouze, who synthesized tetracalcium ferrite hydrate in 1851.

The three anhydrous calcium ferrites mentioned were then hydrated by shaking 1 gram of each of the compounds in 75 cc. of each of three solutions, namely water, half saturated lime water and saturated lime water. The products of hydration were identified by analyses and by X-rays. No optical nor microscopical constants of any of the hydrates are reported and in this respect the data are singularly deficient.

In solutions with a lime (CaO) concentration of 1060 mg per liter or greater, $4CaO.Fe_2O_3.nH_2O$ is stable; in solutions with a lime concentration of less than 640 mg. per liter, hydrolysis occurs and free ferric hydroxide is formed. The compositions of the products of hydration of the calcium ferrites could not be determined in solutions with lime concentrations intermediate between 1060 mg. per liter and 640 mg. per liter.

On hydrating 1g. $2CaO.Fe_2O_3$ in 75 cc. solution the concentration of lime reaches 1060 mg. per liter so that the two tetracalcium ferrite hydrates form.

On hydrating 1g $4CaO.Al_2O_3.Fe_2O_3$ in 75 cc. of solution the concentration of lime is 640 mg. per liter; the end products of hydration are tricalcium aluminate hydrate, a lime-poor ferrite hydrate of unknown composition and free ferric hydroxide.

$CaO.Fe_2O_3$ is practically inert towards the solutions.

A comprehensive review of the literature is included in the report.

Tests of steel columns incased in concrete

A. H. STANG, H. L. WHITTEMORE, D. E. PARSONS. *Journal of Research, National Bureau of Standards*, Vol. 16, No. 3, Mar., 1936, p. 265. HIGHWAY RESEARCH ABSTRACTS

In cooperation with the Port of New York Authority, the National Bureau of Standards tested four carbon-steel columns incased in reinforced concrete to determine their strength and stiffness The results are directly comparable with those for two duplicate steel columns tested by the Bureau without concrete incasement in an earlier series of tests.

The columns were 24 ft. long. The cross-sectional area of the steel was 159 sq. in., and of the concrete, 1245 sq. in. Two of them were loaded on the steel only and two on both steel and concrete. Telemeters were attached to the steel members and the readings indicated that no appreciable stress in the steel was caused by the aging of the concrete. The concrete was about 70 days old when the columns were tested. Measurements were taken of the shortening of the concrete under load, of the lateral deflection of the column, and of the strain in the four horizontal reinforcing rods nearest midheight of the column. When the columns were loaded, the telemeter stresses were always less than the quasi-stress obtained by dividing the load by the cross-sectional area of the steel members. The concrete, therefore, carried a portion of the load.

TABLE 6—STRENGTH OF CASED COLUMNS

Column	Loaded on	Cross-Sectional Area, sq. in.		Column Yield Strength	Final Max. Load	Final Max. Quasi-Stress
		Steel	Concrete	Kips per sq. in.	Kips	Kips per sq. in.
IC 1	Steel only.....	159	1245	51.2	8,278	52.0
IC 2	Steel only.....	159	1245	51.0	8,587	54.0
IC 3	Steel and Concrete..	159	1245	48.6	8,211	51.6
IC 4	Steel and Concrete..	159	1245	50.7	8,179	51.4
Average				50.4	8,314	52.29

NOTE: 1 kip = 1,000 lbs.

The average final max. loads for incased columns was 8,314 kips.

The average final max. loads for unincreased columns was 5,853 kips.

The increase in average final load due to concrete casing was 2,461 kips (42%).

The greatest stress in the reinforcement rods was 10 kips per sq. in.

The lateral deflection was very small until the load on the column approached the maximum. At or about the maximum load, the columns deflected about 6 in. and large pieces of concrete fell from the columns.

Stone sand

A. T. GOLDBECK, Technical Bulletin No. 10 of the National Crushed Stone Association, issued August 1936. (42 pages with illustrations) Reviewed by A. T. GOLDBECK

This bulletin is written with the idea of informing the engineering public and other interested individuals of the characteristics of stone sand, its use, preparation,

and kind of machinery used in its preparation and in procuring the proper gradation. Interesting laboratory test results are given as well as typical illustrations of imposing concrete structures and highways using stone sand as a fine aggregate, showing its successful use in different parts of the country. Stone sand is also used in the manufacture of bituminous mixtures.

After reading the bulletin, the points to be remembered are as follows:

1. Stone sand is an especially prepared product and should not be confused with stone screenings. It can be produced to practically any desired gradation to render it suitable for use as a fine aggregate.

2. Stone sand has been used for the past twenty years and is being used to a greater extent every year and is produced in many different parts of the United States.

3. Its preparation is a comparatively simple operation involving the use of screenings as the parent product. It is desirable that methods of preparation be used which will produce the stone sand as free as possible of flat and elongated particles.

4. All sorts of secondary crushing equipment are used including cone crushers, rolls, hammer mills, etc. Proper gradation is produced with vibratory screens, with water classifying devices and air elutriators. Two methods of preparation are described with an illustration of one method and a drawing of a drag classifier.

5. Laboratory tests are given which indicate exceptionally high beam strength for stone sand concrete. They also show that stone sand concrete can be made equally as durable as natural sand concrete in freezing and thawing tests.

6. Concrete structures of various kinds show stone sand to have been used very successfully in private, State and Federal construction work; some of the structures costing over $1,000,000; typical of these are the Ohio State University Stadium, the Henley Street Bridge at Knoxville, Tennessee, the Norris Dam, railroad tunnels and many miles of concrete highways. Splendid illustrations of such structures are included in the bulletin.

7. There is no difficulty in obtaining a smooth finish, free from honeycomb with stone sand concrete, but a sensible amount of care should be used in the proportioning of the concrete.

8. Stone sand is included in the specifications of the Federal Specifications Board for Concrete Aggregate (SS A—281) for United States Government purchases and by other national and state specifications. A suggested specification for stone sand is included herein.

9. Sheet asphalt made with stone sand has been found to possess exceptional stability.

10. Stone sand is an ideal material for the manufacture of concrete products. In fact, stone sand is well suited for any purpose for which natural sand is used.

The construction of concrete carriageways on the German motor roads

R. Dittrich. *Verkehrstechnik*, 1936, V. 17, No. 6, p. 157-62; No. 7, 185-90. Abstract No. 261, Road *Abstracts*, July, 1936 Highway Research Abstracts

Of the 520 miles of motor roads which had already been opened to traffic in Germany at the end of January, 1936, 485 miles, or 93 per cent, were of cement concrete, bituminous surfacings or small stone setts being used on the remainder. The standard design includes two carriageways, each at least 24 ft. wide, divided by a central strip 12 to 16 ft. 6 in. wide. A margin 40 in. wide is provided between the carriageway and the outer verge, and a 15-in. margin beside the central strip; both margins have a light bituminous surfacing. Each carriageway has a uniform straight camber of 1.5 per cent, super-elevation up to 6 per cent being provided on curves. The

slabs are usually 8 in. thick, but on embanked or excavated sections the thickness may be as great as 12 in. The lengths of the slabs vary from 26 ft. 6 in. to 80 ft.; slabs 100 and 135 ft. long have been installed experimentally. It is usual to vary the length of the slabs in order to prevent periodic vibrations under traffic; thus slabs 48, 56 and 65 ft. long may be placed in that order. A single longitudinal joint is provided in the center of each carriageway, and the transverse joints are generally carried across the entire width. The experimental practice of forming the transverse joints at an angle of 75 deg. with the road axis has been abandoned; at the high speeds for which these roads are constructed, the amount of vibration would not be appreciably affected by the angle of the joints. On bridges and embanked sections, supporting beams are placed below the joints, and on steep gradients the under-surface of the slabs is sometimes ribbed in order to prevent movement; steel channel reinforcement is sometimes provided immediately below the longitudinal joint in order to prevent the separation of adjacent slabs without hindering longitudinal expansion. Dowelled joints have been introduced comparatively recently and will probably be widely used, especially on weak subgrades. Dowels are provided in the lower course concrete only; they are usually spaced 12 in. apart and pass through steel flats 12 in. from the joint. The bars are painted with bitumen and the ends are protected by metal flashing. The concrete is usually placed in two courses. The lower, which is 5.2 to 6 in. thick, contains 450 lb. of cement per cu. yd., with gravel as aggregate; the upper course is 2 to 3 in. thick and contains 504 to 590 lb. of cement per cu. yd., with stone chippings as aggregate. Latterly single course construction has been used; here the concrete contains about 550 lb. of cement per cu. yd., with stone aggregate, gravel being only occasionally used. Reinforcement is used to prevent cracking rather than to increase the bearing power of the material. The most usual type is spot-welded steel mesh, at the rate of 3.8 lb. per sq. yd. Bar reinforcement at the rate of 4.6 to 5.5 lb. per sq. yd. may also be used. In this case the amount of longitudinal reinforcement is about twice as great as that in the transverse direction. Reinforcement is generally placed between the two courses, but on high embankments or weak subgrades a second layer may be employed. At the corners of slabs additional reinforcement is usually provided.

The carbonation of unhydrated cement

Abstract of Technical Paper No. 19 of the Dept. of Scientific and Industrial Research, by D. G. R. BONNELL, M.Sc., Ph.D. *Cement and Cement Manufacture*, Vol. 9, No. 10, Sept. 1936, pp. 195-205.
Reviewed by J. C. PEARSON

Preliminary experiments in which cements were exposed to mixtures of CO_2 and air, both dry and moist, showed that dry CO_2 has no effect on cement, and CO_2—free moist air has very little. On the other hand ignition loss and carbonate content increase rapidly with time of exposure to mixtures of moist air and CO_2. To cements exposed in steel drums a constant flow of 70 per cent moist air with 30 per cent CO_2 was admitted. Stirred thoroughly night and morning, samples taken at 24-h. intervals showed a constant rate of CO_2 absorption until a certain critical value was reached, thereafter a slower but constant rate to the end of the experiment. The amount of CO_2 at the critical point differs with different cements, and bears no relation to the amount of free CaO. The effect of carbonation is to lower the neat tensile strength proportionately to the degree of exposure up to a certain point, then no effect for a while, then steady decline of strength to zero. With mortars, the initial carbonation has no marked effect on tensile strength, and strength persists in mortars even after the neat strength is entirely lost. With fresh cement, a comparatively small degree of carbonation may seriously affect early strength without proving injurious to 28-day strength. Data secured on the rate of carbonation and its effect on

tensile strength suggested that a study of its effect on heat of hydration might develop information on the inflexion points. An abstract of this phase of the investigation is not possible in the limited space available, but interesting confirmations were obtained of the known behavior of the major cement compounds. Thus changes in the rate of carbonation suggested that C_3A was first attacked, and that in the later stages C_3S was mainly involved. At a certain stage a distinct drop in heat of hydration indicated complete decomposition of C_3A. This supports the prevailing notion that C_3A influences strongly the early strengths, but has little or no influence at 28 days. The effect of aeration on setting time is not unidirectional. At high CO_2 content cements invariably develop a fast, if not a flash, set, while with some further carbonation the time becomes very slow. An attempt to explain the variations involves considerable speculation on the effects of compound affinity for water, particle size change and carbonate films over the particles. Certain tests seem to indicate a relation between time of set and the formation and flaking of protective films. A sample of early strength cement was aerated to a CO_2 content of 17 per cent and a loss of 19.6 per cent. The apparent initial set of the cement occurred at 48 hours, and the neat tensile strength at 7 days was zero. Mixed in 1:3 sand mortar the cement gave a strength of 105 p.s.i. at 1 day and 209 p.s.i. at 7 days. This remarkable performance led to a series of mortar tests in which the aggregate was varied in size and concentration. Little strength was attained until a definite size aggregate was reached, the strength then increased with size for a time, and thereafter gradually decreased. The same general course of strength rise and fall was observed with increasing leanness of mix. A somewhat questionable theory is offered in explanation that carbonate films on the cement particles prevent continuity of active material in the neat and rich mixes, and that at a certain size and concentration of aggregate the reduced number of active particles are enabled to form a maximum number of contacts with aggregate particles, which then act as structural links and account for the mortar strength.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY 1937

Portland-Puzzolan Cement as Used in the Bonneville Spillway Dam*

By R. R. Clark† and H. E. Brown, Jr.‡

1—INTRODUCTION

General Description

The Bonneville Project, as it is officially known, consists of a navigation lock, power house, extensive fishways and a spillway dam. An island separates the latter from the other structures, the dam occupying the main river channel.

The method of construction consists of two principal steps in which one half is built in one working season between summer floods. There are about 500,000 yd. of mass concrete and another 100,000 yd. of reinforced concrete. The dam is at Bonneville, Oregon, a small community on the Columbia River, about 150 miles from the Pacific Ocean and virtually at the head of tide water, above which are rapids.

Piers rise 75 ft. above the crest of the dam and support 18 vertical lift gates, 54 ft. long, some of which are 60 ft. high and the remainder 50 ft. high.

Location and Foundation Conditions

The location is in the heart of the Columbia River Gorge, where it cuts the Cascade Mountains at right angle—the mountain range which extends about 500 miles from the Siskiyous on the south to the Selkirks on the north.

The bedrock underlying the spillway dam is volcanic in origin and belongs to the "Eagle Creek" formation. It is made up of three different rock types: (1) conglomerates of rounded, andesitic pebbles up to 6-in. diameter in a matrix of rock fragments, sand, and clay-

*Received by the Institute Jan. 13, 1937 for presentation 33rd Annual Convention.
†Designing Engineer of the Spillway Dam, U. S. Engineer Office, Portland, Oregon.
‡Assistant Engineer.

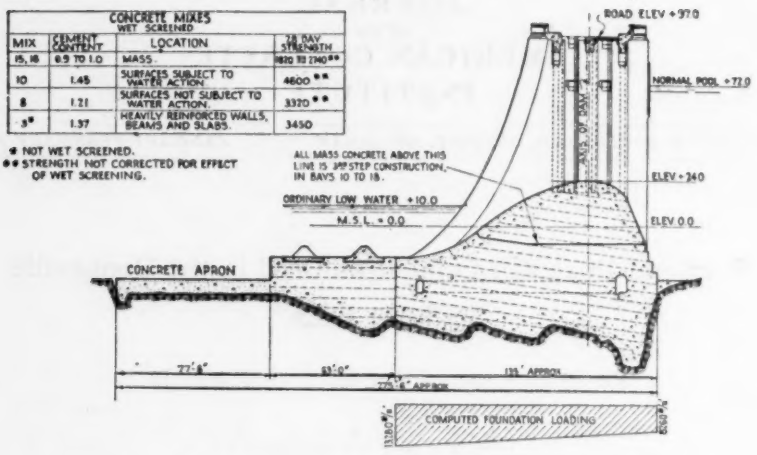


FIG. 1—GRAVITY SECTION OF SPILLWAY DAM

like material, derived from the original material by hydration; (2) sandstone composed of angular mineral and rock fragments, and united with the same matrix; and (3) the so-called "bentonite," composed largely of this clay-like hydration product but usually with more or less admixed sand. There is a wide variation and gradation, but all types are well consolidated, and have a hardness between that of sandstone and shale. The dip of the rock is about 15 deg. southeast, or upstream, which is favorable to prevent sliding and shear.

Design of Dam

By reference to Fig. 1 it is evident that the proportions are quite different from other large dams, in that the ratio of its base width to height is large. Calculated compressive stresses are low. The dam is divided into blocks 60 ft. long by contraction joints to be grouted. Downstream a vertical joint parallel with the axis and 97 ft. distant divides the gravity section from the baffle deck. This joint is also to be grouted. Specifications call for mass concrete to be placed in 5 ft. lifts and an interval of three days to elapse between lifts.

In the original investigation two designs were considered, one of which provided vertical contraction joints parallel with the axis and so designed to transmit shear through the joint and become more positive on subsequent grouting. This design was ultimately rejected for a design without such contraction joints, but with provision for a special type of cement from which it was thought concrete could be

produced that in itself would meet the requirements of design and to a large extent prevent cracking.

Practical considerations played no small part in this decision. So far as is known, a dam has not yet been built in a river of equal flow and in which the force of the current is as great. It was felt therefore that anything that would increase the time of actual work within the cofferdam for each season should be eliminated as far as possible. To have provided the contraction joints would have meant building 45 instead of 27 individual blocks in each season's work.

The decision therefore greatly increased the importance of cement and concrete investigations which supplied data concerning the type of cement under consideration. Mass concrete problems have been greatly increased by modern methods of construction which have been developed to meet the demand for concrete in large masses. A significant statement occurs in a recent publication by J. L. Savage, and others of the Bureau of Reclamation, which is: "In some instances it is possible materially to reduce the initial cost and at the same time realize quality by use of the proper type of cement."

2—CEMENT INVESTIGATION AND SELECTION

In the interest of low thermal changes, it was considered desirable to employ a cement for which the ultimate heat of hydration would be reasonably small but which would generate a large part of its heat of hydration during the early hardening period in order that as much as possible of the heat might be dissipated from each lift to the air before the lift would be covered by the succeeding lift. Also, it was considered desirable that the cement content be as low as consistent with the requirements of early strength and impermeability. The spillway section will seldom be exposed to drying and then only for short periods. A low contraction coefficient on drying was therefore not of primary importance.

In the light of previous experience, it was considered particularly desirable that the cement be of a character that would produce in lean mixes a plastic workable concrete exhibiting a minimum tendency toward segregation and water gain.

Requirements of Concrete

Under the generally prevailing conditions of low placement temperature, it was considered necessary that the concrete have reasonably good early strength in order that forms could be removed at an early age. Since the calculated stresses due to load on the dam were small, high ultimate compressive strength of concrete was not a primary requisite; but high tensile strength was considered desirable

in order that the structure might better resist without cracking the stresses incidental to temperature changes within the concrete mass.

Types of Cement Considered

Having in mind the design features of Bonneville Dam and the particular conditions surrounding its construction and use, it appeared that there were possibilities of producing concrete of the desired properties with a fairly finely ground cement of one of the following three types:

Type A: A portland cement low in tricalcium aluminate but normal in composition as regards tricalcium silicate; similar to that employed in the construction of Norris and Tygart Dams and the Colorado River Aqueduct. In current literature, this type of cement is often referred to as "modified" portland cement.

Type B: A portland cement low in tricalcium aluminate and high in dicalcium silicate; similar to that used in the construction of Morris and Boulder Dams. In current literature, this is termed a "low-heat" portland cement.

Type C: A portland-puzzolan cement produced by intergrinding a portland cement of the modified type with a puzzolan. A puzzolan is defined as a siliceous material that in the presence of moisture will combine with lime to form an insoluble strength-producing compound. During the hydration of portland cement, lime is liberated. During the hydration of portland-puzzolan cement, the lime liberated by the portland cement constituent is considered to combine with the puzzolan. Portland puzzolan cements have been extensively used in Europe for hydraulic structures. Recent examples of American construction in which cements of this type have been employed are certain massive piers of the Golden Gate Bridge and San Francisco-Oakland Bay Bridge.

Investigations Prior to Purchase of Cement

An investigation of the three types of special cement—modified portland, low-heat portland, and portland-puzzolan—was undertaken for the War Department by the University of California in December 1934. The cements were manufactured in the Engineering Materials Laboratory of the University. The portland-puzzolan cements (Type C) contained 25 per cent of puzzolan. The puzzolans included calcined Monterey shale, calcined diatomaceous earth, calcined pumicite, and calcined clay. Concretes of two cement contents were used in the tests, 0.8 and 1.0 bbl. per cu. yd. The aggregates were from the same supply as that used in the construction of the Bonneville power house and locks; the maximum sizes of aggregate were 1½, 3, and 6 in. For a given test, the concretes containing the various cements were of fixed consistency. For many of the tests the specimens were cured under simulated mass-concrete temperature conditions from a placement temperature of 40° F. The following properties of cement, mortar, or concrete were determined under various conditions of test: Water requirement; water gain; permeability; heat of hydration and temperature rise; thermal properties; strength and elasticity in

TABLE 1—CHEMICAL AND PHYSICAL PROPERTIES OF CEMENTS

		Type A (Modified Portland	Type B (Low-heat Portland	Type C (Portland-pozzolan)				
				Shale	Diat. Earth	Pumicite	Clay	Av.
Compound composition of clinker, per cent	3CS	51	26	52	52	52	52	52
	2CS	22	46	21	21	21	21	21
	3CA	6	6	6	6	6	6	6
	4CAF	17	16	17	17	17	17	17
Fineness	% pass. No. 200 sieve	97.7	98.0	96.7	97.4	98.3	98.2	97.7
	% pass. No. 325 sieve	89.1	91.3	90.5	89.1	90.9	92.1	90.7
	Sp. surf., sq. cm per g.	1950	1980	2130	2170	2170	2290	2190
Specific gravity		3.18	3.19	2.92	2.96	2.93	3.00	2.95
Percentage of water for normal consistency		22.9	24.5	37.5	32.0	25.5	28.5	30.9
Tensile strength of standard mortar, p.s.i.	1 da.	210	65	175	185	145	150	165
	3 da.	370	200	315	340	270	255	295
	7 da.	420	275	395	435	345	370	385
	28 da.	440	410	540	525	470	475	505
	3 mo.	480	475	590	565	560	500	555
	5 mo.	470	495	615	555	565	495	560

compression and tension; volume changes due to drying and to changes in temperature; plastic flow under sustained load; and extensibility. The total number of specimens was about 4060 and the number of tests considerably more.

Test Results—The chemical and physical properties of the three types of cement are shown in Table 1; and the results of certain of the tests are shown in Tables 2 to 4. In order to include as many test results as possible within a limited space and to reduce the irregularities of test, the reported values in as many cases as possible represent average results for two finenesses of cement or two cement contents of concrete. The values of specific surface shown in the tables were determined by the hydrometer method,* and are not comparable with values which would be obtained under the present A. S. T. M. standard, C115-34T.

In Table 1, it is seen that as compared with the portland cements the portland-puzzolan cements were of lower specific gravity and required more water for normal consistency of neat-cement paste. The portland-puzzolan cements characteristically exhibited tensile strengths of standard mortar which at the early ages were greater than those for the low-heat portland cement and at the later ages were greater than those for either of the portland cements.

The water requirement of the various cements for concrete of a fixed consistency is shown in Table 2. The portland-puzzolan cements containing shale and diatomaceous earth required more water than

*"A Hydrometer Method for Determining the Fineness of Portland-Puzzolan Cements," by S. B. Biddle Jr. and Alexander Klein, A. S. T. M., 1936.

TABLE 2—WATER REQUIREMENT, WATER GAIN, AND PERMEABILITY OF CONCRETES

	Type A (Modified Portland	Type B (Low-heat Portland	Type C (Portland-puzzolan)				
			Shale	Diat. Earth	Pumicite	Clay	Av.
Water-cement ratio, by wt., required for ¾-in. slump of concrete (Cement content 1.0 bbl. per cu. yd.; 0 to 1½ in. aggregate)	0.68	0.67	0.73	0.70	0.66	0.65	0.68
Water gain, milliliters, on 18 by 36-in. concrete cylinder (Cement content 0.9 bbl. per cu. yd.; 0 to 6-in. aggregate; slump of wet-screened concrete 1½ in.)	135	63	29	—	—	—	—
Inflow of water, milliliters, into 30 x 30-in. concrete cylinder during period of 36 hr. under pressure of 100 p.s.i. (Cement content 0.9 bbl. per cu. yd.; 0 to 6-in. aggregate; slump of wet-screened concrete 1½ in.; standard curing; specimens tested at age of 28 days)	480	320	180	—	—	—	—

the portland cements, but the portland-puzzolan cements containing pumicite and clay require less water than the portland cements.

At the time of manufacture of specimens, it was observed that concretes containing the portland-puzzolan cements were markedly more plastic and workable, and tended less to segregate, than those containing the portland cements. As between the two portland cements, the difference in workability was small.

The ability of a cement to retain the mixing water during the setting period is a property of considerable importance in producing homogeneous concrete. In this investigation, the water rising to the surface of 18 by 36-in. concrete cylinders during the first 2 hours after casting was collected by means of blotters and was weighed. Table 2 shows that the water gain for the portland-puzzolan cement was only about one fourth as great as that for the modified portland cement and only one half as great as that for the low-heat portland cement.

The permeability of mass concretes containing the three types of cement was determined by measuring the inflow of water under pressure into large specimens of concrete. The water was introduced through a porous mortar cylinder at the center of the specimen. Table 2 shows that the inflow of water for the portland-puzzolan cement was roughly one third of that for the modified portland cement and roughly one half of that for the low-heat portland cement.

As shown in Table 3, the heat of hydration at 70°F. of the portland-puzzolan cements was less than that of the modified portland cement at all ages up to 28 days. On the average, the portland-puzzolan

TABLE 3—HEAT OF HYDRATION OF CEMENTS

			Type A (Modified Portland	Type B (Low-heat Portland	Type C (Portland-puzzolan)				
					Shale	Diat. Earth	Pumicite	Clay	Av.
Heat of hydration, calories per gram	70°F.	1 da.	38	25	36	35	32	32	34
		3 da.	58	51	55	54	50	51	52
		7 da.	75	61	67	68	61	64	65
		28 da.	85	68	78	79	71	77	76
	40°F.	1 da.	16	14	18	—	—	—	—
		3 da.	36	25	37	—	—	—	—
		7 da.	50	40	50	—	—	—	—
		28 da.	76	64	65	—	—	—	—

Note:—Water-cement ratio 0.40 by wt.; vane calorimeter; values represent heat generated from time specimen was placed in calorimeter, 15 min. after cement was wetted.

TABLE 4—COMPRESSIVE AND TENSILE STRENGTHS OF CONCRETE

			Type A (Modified Portland	Type B (Low-heat Portland	Type C (Portland-puzzolan)				
					Shale	Diat. Earth	Pumicite	Clay	Av.
Water-cement ratio, by wt.			0.68	0.67	0.73	0.70	0.66	0.65	0.68
Standard Curing	Compressive strength, p.s.i.	3 mo.	3600	3750	3790	4150	3500	3270	3680
		5 mo.	3870	4240	4150	4180	3780	3300	3850
Mass Curing from 40°F.	Compressive strength, p.s.i.	1½ da.	170	50	210	....	....	....	....
		3 da.	610	170	790	610	440	600	610
		7 da.	1620	530	1760	1480	1100	1430	1440
		28 da.	3300	2640	3540	3220	2610	2890	3060
		3 mo.	3830	4090	3810	3710	3690	3410	3650
		5 mo.	4030	4650	4050	3720	4230	3260	3810
	Tensile strength, p.s.i.	5 da.	110	40	120	....	....	....	....
		14 da.	180	140	200	....	....	....	....
		28 da.	270	230	340	....	....	....	....
		3 mo.	300	290	350	....	....	....	....
		5 mo.	280	300	370	....	....	....	....

Note:—Concrete contains 1.0 bbl. of cement per cu. yd.; 0 to 1½-in. aggregate; slump ¾ in.

cements generated more heat at 70° F. than the low-heat portland cement, but the difference in heat generation as between the portland-puzzolan cement containing pumicite and the low-heat portland cement was not great at ages of 3 to 28 days. At the control temperature of 40° F., the portland-puzzolan cement containing shale exhibited a considerably higher ratio (0.57) of 3-day to 28-day heat of hydration than did the modified portland cement (0.47) or the low-heat portland cement (0.40).

Compressive strengths of concretes containing the various cements are shown in Table 4. Under the condition of mass curing from 40°F., the low-heat portland cement exhibited very low compressive strengths

of concrete at early ages, but excellent strengths at later ages. On the average, under either of the two curing conditions, the compressive strengths of concrete did not differ greatly as between the modified portland and the portland-puzzolan cements.

At all ages of test, the tensile strength of concrete was higher for the portland-puzzolan cement than for the portland cements (Table 4).

In a separate investigation the volume changes of portland-puzzolan cements containing the various puzzolans were determined. The following tabulation shows the relative contraction of mortar bars after 5 months of drying in air of 50 per cent relative humidity at 70° F.

Cement		Percentage of Net Contraction of Mortar Containing High-Lime Portland Cement
High-lime portland (59% 3CS, 11% 3CA)		100
Low-lime portland (37% 3CS, 11% 3CA)		135
Portland-puzzolan (75% high-lime portland, 25% puzzolan)	Shale	169
	Diat. earth	140
	Pumicite	125
	Clay	109
	Average	136

It is seen that the portland-puzzolan cements exhibited considerably greater contraction than the high-lime portland cement, but that on the average the contraction was about the same for the portland-puzzolan cements as for the low-lime portland cement which was most nearly comparable to the low-heat portland cement of the Bonneville Dam cement investigation.

Plastic flow as measured by the sustained modulus of elasticity (unit stress divided by the sum of the elastic and plastic unit deformations) of concrete made from the *C* cement was higher than that of the *B* cement, but lower than the *A* cement. At later ages the sustained modulus of the *C* cement was lower than for either of the other two cements, which means a higher plastic flow coefficient.

Selection of Cement

In addition to the results of the investigation just described, there were available the results of previous investigations and of construction experience. It appeared that a low-heat portland cement could not be employed because when cast at low temperatures its very low early strengths would have seriously delayed the removal of forms. Examinations of existing mass-concrete structures in which modified

portland and portland-puzzolan cements had been employed led to the conclusion that the tendency toward serious cracking due to thermal changes was considerably less for the concretes containing portland-puzzolan cements. Further considerations which resulted in the selection of a portland-puzzolan cement over a modified portland cement, as disclosed by test results, were lower heat of hydration, higher tensile strength, greater impermeability, greater workability, greater freedom from segregation, and lower water gain.

3—SPECIAL CEMENT SPECIFICATIONS

The specifying of the properties of a suitable portland-puzzolan cement involved several new problems, inasumch as standard specifications had not been established in America and as European specifications did not appear to be adequate. In order to obtain portland-puzzolan cement of the desired characteristics, it was considered necessary to establish certain chemical requirements for the clinker and chemical and physical requirements for the puzzolan, as well as chemical and physical requirements for the finished cement. Methods of chemical analysis formerly used or recently proposed were extended as necessary to include the analysis of puzzolanic materials.

Specifications and Acceptance Tests

It was desired to employ an acceptance test for the puzzolan which would indicate its activity, or ability to combine with lime. For this purpose there was employed a test of puzzolan-lime-sand mortar which consisted in determining the 7 and 28-day compressive strengths of 2 by 4-in. cylinders of mortar containing 2 parts by weight of puzzolan of stated fineness, 1 part of hydrated lime of stated composition and fineness, and 9 parts of standard Ottawa sand. The water content of the mortar was determined by the method used for standard mortar containing portland cement. The cylinders were cured in sealed metal cans, at 70° F. for 12 hr., then at 100° F. for 12 hr., then at 130° F. until 12 hr. before the time of testing, and then at 70° F.

Another special problem was the determination of fineness of portland-puzzolan cement. Satisfactory turbidimetric methods had been developed for portland cements which were opaque; but as puzzolans were translucent to various degrees these methods were not generally applicable to portland-puzzolan mixtures. A sedimentation test employing a sensitive hydrometer was adopted from that used in the measurement of fineness of soils. Briefly, the method of test was as follows: A sample of portland-puzzolan cement was dispersed in kerosene, in a tall cylinder. As the cement settled, at intervals the hydrometer was inserted and the specific gravity of the suspension

TABLE 5—COMPARISON OF AVERAGE RESULTS OF TESTS ON BONNEVILLE DAM PORTLAND-PUZZOLAN CEMENT WITH SPECIFICATION REQUIREMENTS

		Specification Requirement	Approximate Average (1935-1936 Season)
Clinker	Sulfuric anhydrid (SO_3), per cent	max. 0.50	0.5
	Magnesia (MgO), per cent	max. 5.00	1.7
	Insoluble residue, per cent	max. 0.85	0.1
	Ratio of iron oxide (Fe_2O_3) to alumina (Al_2O_3), per cent	max. 1.56	1.0
	Tricalcium silicate, per cent	max. .55	50
	Tricalcium aluminate, per cent	max. 7	5
Puzzolan	Silica (SiO_2), per cent	min. 50.00	68
	Lime (CaO), per cent	max. 10.00	3.5
	Loss on ignition, per cent	max. 3.00	0.1
	Compressive strength of puzzolan-lime-sand mortar at 7 da., p.s.i.	min. 500	660
	Ditto, at 28 da. p.s.i.	min. 800	910
Portland Puzzolan Cement	Loss on ignition, per cent	max. 3.00	0.6
	Sulfuric anhydride (SO_3), per cent	max. 2.00	1.3
	Magnesia (MgO), per cent	max. 5.00	1.6
	Percentage passing No. 325 sieve (wet)	min. 88	93
	Specific surface, sq. cm. per gram	min. 1800	1950
	Time of initial setting, hr. : min.	min. 1:00	2:20
	Time of final setting, hr. : min.	max. 10:00	4:00
	Tensile strength of standard mortar at 7 da., p.s.i.	min. 275	320
	Ditto, at 28 da., p.s.i.	min. 350	410

Note:—Specific gravity of portland-puzzolan cement 3.03.; 24 per cent of water required for normal consistency of neat-cement paste.

at the level of the hydrometer bulb was measured. By means of Stokes' law pertaining to bodies falling in a viscous medium, the percentages of various size fractions of cement were calculated; and from these data the surface areas of the fractions were calculated and summed to obtain the total surface area.

The specifications prescribed in considerable detail the properties of the portland-cement clinker, the puzzolan, and the finished cement. It was required that the clinker be of the modified portland type, with the tricalcium aluminate limited to a maximum of 7 per cent. It was specified that the puzzolan contain not less than 50 per cent of silica, that it be calcined, and that it meet the requirements of the compression test for activity. With regard to the finished cement, the specification requirements for sulfuric anhydride, magnesia, soundness (steam test), time of setting, and tensile strength were the same as those of the current A. S. T. M. specification for portland

cement. The minimum fineness of finished cement was prescribed by the requirements that the specific surface should be not less than 1800 sq. cm. per gram and that not less than 88 per cent should pass the No. 325 sieve; the sieve requirement was added because otherwise the specific-surface requirement might be met by blending an extremely fine puzzolan with a coarse portland cement. The ordinary steam test for soundness was supplemented by the Merriman test for resistance to sodium sulfate of a neat-cement slab.

4—FINAL CONCRETE STUDIES

As soon as cement and aggregate became available in the summer of 1935 an extensive program of mixture design experiments was conducted.

Characteristics of Cement as Purchased

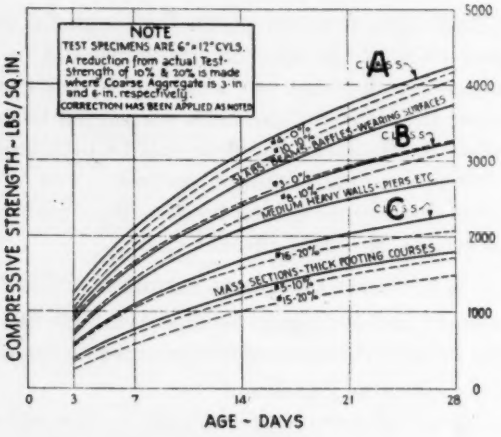
Average results of laboratory tests made in the course of routine inspection for acceptance are shown in Table 5. The tests cover the construction period from Sept. 1935 to May 1936.

The heat of hydration at 70° F., determined by means of the vane calorimeter, was considerably less for the Bonneville Dam portland-puzzolan cement than for any of the portland-puzzolan cements previously tested. The following tabulation affords a comparison with the average of the portland-puzzolan cements containing shale, diatomaceous earth, pumicite, and clay.

Cement	Heat of Hydration at 70° F., Calories per Gram including Immediate Heat of Hydration			
	1 Da.	3 Da.	7 Da.	28 Da.
Bonneville Dam portland-puzzolan	32	49	63	68
Average for four portland-puzzolan cements shown in Table 3	38	56	69	80

Trial mix tests

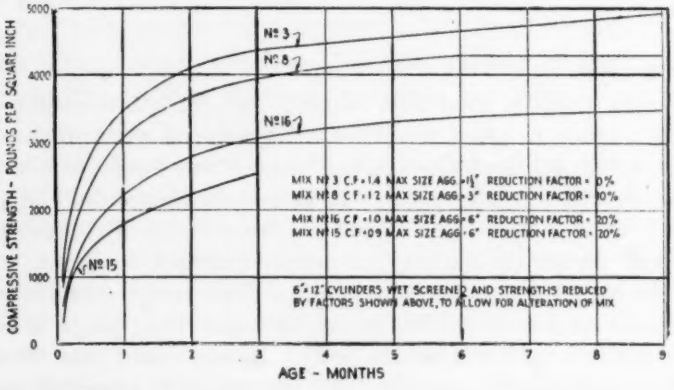
Aggregate supplies were first studied and gradation curves established as a guide to plant operations. Specimens were prepared and tested in which the first shipment of cement was used. While it was expected that the greater part of the mass concrete would be placed in winter months, these trial mix tests were made in August but at the coldest period in the day, namely, midnight to dawn. These tests were made in part at Bonneville and in part at the University of California and included 105 mixes having a wide range of cement content (0.75 to 1.80 bbl. per cu. yd.). It was found that there was an optimum of cement content of 1.5 bbl. which if exceeded resulted in crazing or surface cracking. The workability was rated by visual inspection at the time of mixing and molding. The results of this set



MIX No.	CLASS SYMBOL	C/F BBL/CU YD	SAND/GRAVEL RATIO	WATER CEMENT RATIO	SLUMP (Field)	MIXER BATCH (LBS) CEMENT	SAND	GRAVEL	WATER	YIELD CU. YDS.
1	BM-0	2.71	100-1	0.80	10"	2040	4410		1080	2.00
3	B-2	1.37	40-60	0.90	4"	2070	5310	7950	1240	4.02
4	A-2	1.55	40-60	0.80	5"	2390	5300	7970	1260	4.09
5	C-3	0.97	35-65	1.10	3"	1480	5100	9460	1080	4.07
8	B-3	1.20	35-65	0.90	3"	1830	4940	9160	1090	4.05
10	A-3	1.44	35-65	0.80	3"	2180	4740	8800	1160	4.04
15	C-4	0.87	30-70	1.15	3"	1360	4530	10560	1040	4.14
16	C-4	1.00	30-70	1.00	2½"	1520	4400	10270	1010	4.06

CONCRETE CLASSIFICATION FOR STRENGTHS AS ADOPTED FROM TRIAL MIX TESTS

FIG. 2—CONCRETE CLASSIFICATION FOR STRENGTHS AS ADAPTED FROM TRIAL MIX TESTS



CYLINDERS FROM CONCRETE PLACED TO DEC. 1936

FIG. 3—COMPRESSIVE STRENGTH FROM ROUTINE TESTS

of experiments made possible the adoption of a strength classification chart, which is shown as Fig. 2.

Owing to the lower tricalcium silicate content of the average cement used in construction as compared with the cement used in the trial mixes, the 7-day compressive strengths of concrete were lower for the field mixes than for the corresponding trial mixes. The results of trial mix tests for water gain and workability compare favorably with the results of the early investigations. This is further borne out by field conditions in which it is continually noted that in both lean and rich mixtures there is an absence of water gain or bleeding. It is to be noted that the lower specific gravity of the puzzolan cement results in a greater bulk volume of cement per barrel, consequently a more plastic mixture. This also improves the property of impermeability since the puzzolanic materials when interground with portland cement clinker evidently grinds to a greater fineness than the clinker. This is proven by the fact that in screen analysis the residue is invariably portland cement

Routine Testing

Test cylinders taken from job mixes at regular and frequent intervals are cured by standard curing methods. The data obtained to date from routine testing is reproduced in Fig. 3, and includes a total number of specimens tested to date, about 3,000 in all. The strengths for the second season are consistently higher than those for the first season. One thing that doubtless contributes to this higher strength is the reduction in quantity of No. 4 to ⅜ in. material in the coarse aggregate. 1936 test records are not yet sufficiently complete to be treated separately. The principal mixes used throughout are shown in Table 6 which are again first season 1935 or values.

All cylinders are 6 x 12 in. and the mixes containing coarse aggregate larger than 1½ in. are wet screened through a 2-in. sieve. The strengths of mixes containing 6 in. maximum aggregate have been reduced by 20 per cent and those containing 3 in. maximum size aggregate by 10 per cent to compensate for the alteration of the mix wet screening. The 28-day strengths for mixes No. 8 and 16 are within the shaded section of the strength classification chart of the trial mix tests. The strength of mix No. 3 is slightly higher than its design strength. Also to be noted is the relatively large gain in strength after 28 days, this is a characteristic of concretes made from portland-puzzolan cements. The ratios of the six months to the 28-day strengths are 1.33, 1.46, and 1.61 for mixes No. 3, 8, and 16 respectively.

TABLE 6—COMPRESSIVE STRENGTH OF CONCRETES USED IN CONSTRUCTION DURING SEASON OF 1935-1936

Mix No.		15	16	8	10	3
Location of Concrete in Structure		Interior of Spillway Dam During Warm Weather	Interior of Spillway Dam During Cold Weather	Surfaces of Spillway Dam Not Subject to Erosion; Interior of Large Piers; Thick Walls	Surfaces of Spillway Dam, Apron and Piers Subject to Erosion	Heavily Reinforced Walls, Beams, and Slabs
Maximum size of aggregate, in.		6	6	3	3	1½
Ratio of sand to coarse aggregate, by wt.		30:70	30:70	35:65	35:65	40:60
Cement content, bbl. per cu. yd.		0.88	1.00	1.21	1.45	1.37
Water-cement ratio, by wt.		0.75	0.63	0.58	0.52	0.60
Slump at mixing plant, in.		3½[a]	4¼[a]	3½[a]	2¾[a]	4½
Compressive strength, p.s.i. (6 by 12-in. cylinders, standard curing)	3 da.	390[a]	700[a]	900[a]	1460[a]	940
	7 da.	920[a]	1360[a]	1780[a]	2630[a]	1870
	28 da.	1870[a]	2740[a]	3320[a]	4600[a]	3450
	3 mo.	....	3810[a]	4370[a]	....	4360
	6 mo.	....	4210[a]	4870[a]	....	4550
	9 mo.	....	4360[a]	....	....	....

[a]—Concrete wet-screened. Strengths not reduced.

In addition to the trial mix tests, certain other tests were made at the University of California to determine the following: Shrinkage of mortars and concrete, temperature rise of adiabatically cured concrete, thermal properties, and plastic and elastic properties of concretes at early ages.

Drying Shrinkage

Tests made on mortar bars to compare shrinkage of the job cement with that of the *A* and *C* cements of the earlier investigation showed that at the age of 135 days, after being moist cured for 14 days and then stored in air at 70° F. and 70 per cent R. H., the specimen made from the job cement had contracted about 20 per cent more than those made of the *A* cement and 26 per cent less than those made of the *C* cement. Shrinkage tests were also made on concrete bars 3 by 3 by 40 in. and on a concrete prism 2 by 2 by 3 ft. Within the prism Carlson strainmeters were embedded parallel and transverse to the longitudinal axis of the prism, also strain gage plugs were provided on the 2 by 2 ft. surfaces. At the age of 15 days the four 2 by 3 ft. faces were painted with three layers of coal tar cutback leaving the ends exposed and simulating the drying conditions in a wall exposed on two sides. The bars at an age of 170 days contracted more than two and one-half times as much as the exposed surfaces of the prism. The strainmeters showed that the internal shrinkage of the prism was about one-half that at the surface.

Adiabatic Temperature Rise

The adiabatic temperature rise of a mass mix containing 0.9 bbl. of cement per cu. yd. and 0 to 6 in. aggregate was determined from 12 x 14 in. cylinders with sensitive resistance thermometers embedded for temperature control and measurement. The specimens were cured in sealed cans in an insulated adiabatic calorimeter room equipped for precise temperature regulation. The temperature of the room was automatically regulated to be the same as the temperature of the specimen. The initial curing temperatures were 40, 50, 60, and 70° F. The results of these tests are plotted in Fig. 4 which shows that at the early ages, the lower the placement temperature the lower the temperature rise, but at the age of 28 days the lower the placement temperature the higher the temperature rise. This relation has been observed in tests on other cements.

Thermal Expansion and Contraction

The results of tests to determine thermal expansion and contraction coefficients gave these values: 0.000004 per degree F. for the first day and 0.000005 per degree F. thereafter.

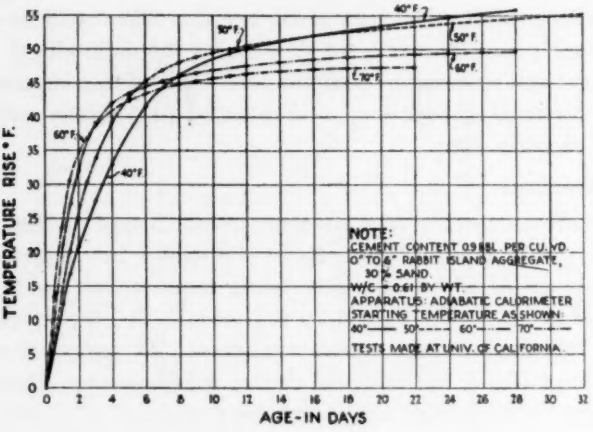

TEMPERATURE RISE °F.
AGE - IN DAYS
80° F.
40° F.
50° F.
60° F.
70° F.
NOTE:
CEMENT CONTENT 0.9 BBL. PER CU. YD.
0" TO 6" RABBIT ISLAND AGGREGATE,
30 % SAND.
W/C = 0.61 BY WT.
APPARATUS: ADIABATIC CALORIMETER
STARTING TEMPERATURE AS SHOWN:
40° —— 50° - - - - 60° ····· — 70° - · - · -
TESTS MADE AT UNIV. OF CALIFORNIA.

FIG. 4—ADIABATIC TEMPERATURE RISE FOR VARIOUS PLACING TEMPERATURES

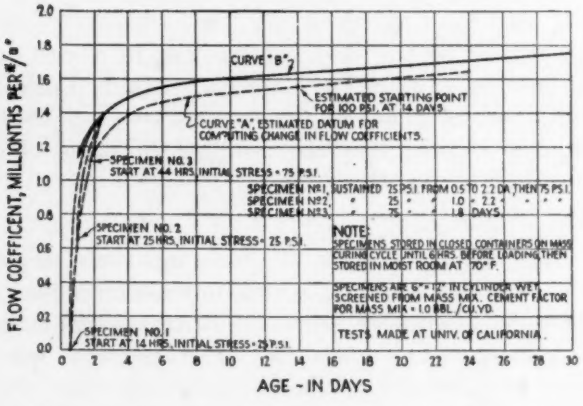

FLOW COEFFICENT, MILLIONTHS PER #/□"
AGE - IN DAYS
CURVE "B"
ESTIMATED STARTING POINT FOR 100 PSI. AT 14 DAYS
CURVE "A", ESTIMATED DATUM FOR COMPUTING CHANGE IN FLOW COEFFICIENTS.
SPECIMEN NO. 3 START AT 44 HRS. INITIAL STRESS = 75 P.S.I.
SPECIMEN NO. 2 START AT 25 HRS. INITIAL STRESS = 75 P.S.I.
SPECIMEN NO. 1 START AT 14 HRS. INITIAL STRESS = 75 P.S.I.
SPECIMEN NO 1, SUSTAINED 75 P.S.I. FROM 0.5 TO 2.2 DA. THEN 75 P.S.I.
SPECIMEN NO 2, " 75 " " 1.0 - 2.2 " " " "
SPECIMEN NO 3, " 75 " " 1.8 DAYS.
NOTE:
SPECIMENS STORED IN CLOSED CONTAINERS ON MASS CURING CYCLE UNTIL 6 HRS BEFORE LOADING, THEN STORED IN MOIST ROOM AT 70° F.
SPECIMENS ARE 6" × 12" IN CYLINDER WET SCREENED FROM MASS MIX. CEMENT FACTOR PER MASS MIX = 1.0 BBL. / CU. YD.
TESTS MADE AT UNIV. OF CALIFORNIA.

FIG. 5—PLASTIC FLOW COEFFICIENT DIAGRAM

Plastic Flow Coefficients

From the results of tests made to determine the plastic characteristics at early ages of concrete containing one barrel of cement per cu. yd. curves were plotted of flow per unit of stress against time. These curves for cylinders loaded at different ages had essentially the same shape after the load had been applied for several days although the amount of flow was different for each age of loading. To plot the "B" curve, Fig. 5, the curve of the cylinder loaded at the earliest age was plotted starting with a flow coefficient of zero. The curves for specimens No. 1, 2 and 3 were plotted from this curve by plotting

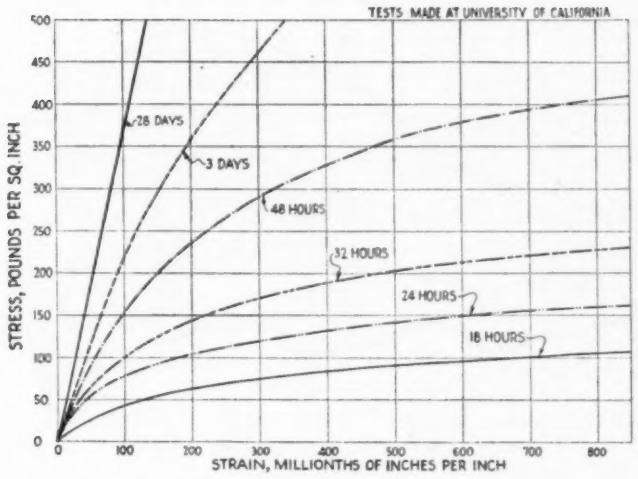


FIG. 6—STRESS STRAIN RELATIONS OF CONCRETE

the flow coefficient at the oldest age on the curve of the specimen No. 1. The remaining values of flow coefficient were plotted with reference to the plotted position of the value at the oldest age.

This procedure left the starting points of the several curves below the curve for the specimen No. 1 loaded at the earliest age and above the zero ordinate of flow coefficient. The "*A*" curve was next drawn through the starting points of all the specimens.

The amount of flow caused by a load applied at a given age for a given length of time is the product of the unit stress multiplied by the difference between the ordinate of the "A" curve at the age the load is applied and the ordinate to the "B" curve at the age the load is removed.

Modulus of Elasticity

Cylinders, 6 x 12 in., made from the same concrete as used for the plastic flow specimens were tested for modulus of elasticity at early ages. The cylinders were tested at the ages of 18, 24, 32, 48 hrs., 3 days and 28 days. Deformations were measured with a single-dial-gage compressometer of conventional type having a gage length of 8 in. Fig. 6, shows the stress-strain relations determined by these tests.

5—EMBEDDED INSTRUMENTS

Measured temperatures and strains have an important place in studies of volume changes in mass concrete and for a guide in attempts at rational analyses of stresses and strains in mass concrete. Carlson

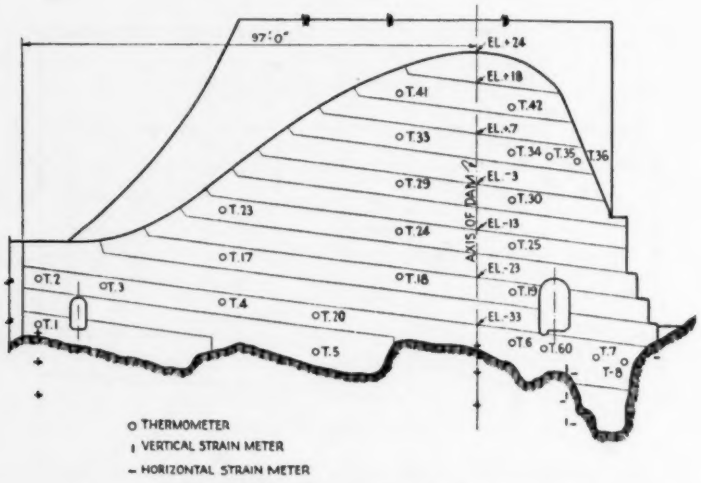


FIG. 7—INSTRUMENT LOCATION DIAGRAM BLOCK 10

electrical resistance thermometers were embedded in the foundation and concrete of blocks 9, 10, and 14 of the first season's construction. Fig. 7 shows the location of the thermometers in block 10 and is typical for blocks 9 and 14. Carlson elastic wire strainmeters were embedded in the foundation and concrete of block 10 to measure vertical and horizontal deformations of the rock. The location of these instruments is also shown by Fig. 7.

Temperature Records

Fig. 8 shows the observed temperature at various points in blocks 9, 10 and 14.

Isothermic lines for block 10 on Nov. 1, 1935, and Sept. 1, 1936, are shown in Fig. 9. The average temperature of the block reached a maximum on Nov. 1, 1936 with 95° near the center of the mass. Ten months later the center temperature had fallen to 75°, Fig. 10 shows the variation in temperature in two lifts of block 14. An abrupt fall in air temperature of nearly 20° F. on the day following the placing of the lower of the two lifts caused a greater range of temperature within this lift than was usually the case, and resulted in a larger rise of temperature at the surface when the succeeding lift was placed. The low air temperature and seven-day interval between lifts resulted in a low average maximum temperature rise of about 27° for this lift. Fig. 11 is a study of the effect of the period of exposure, placing and air temperatures on the rise in temperature of a 5 foot interior lift. Comparing curves 4 and 5 for which the time of exposure is nearly

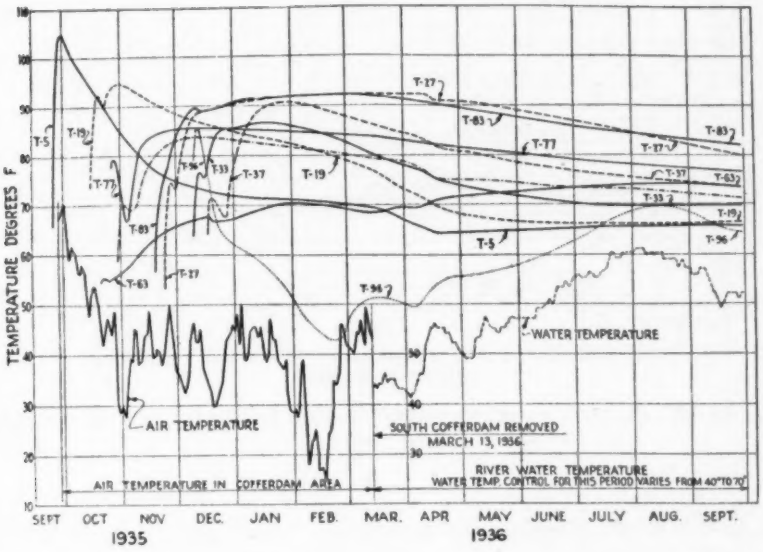


FIG. 8—MEASURED TEMPERATURES SELECTED FOR BLOCKS 9, 10 AND 14

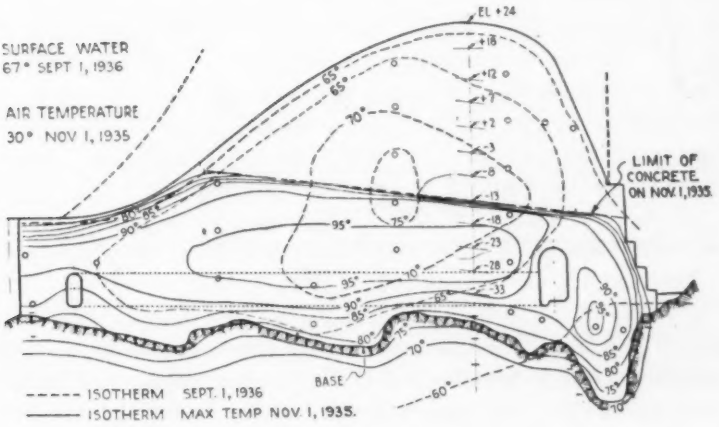


FIG. 9—ISOTHERMS BLOCK 10

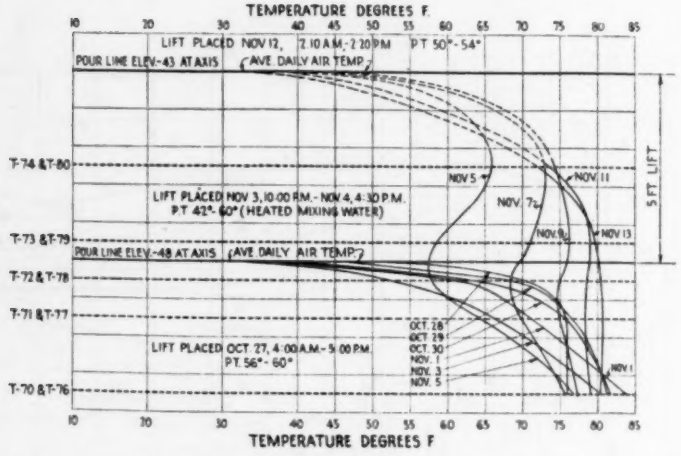


FIG. 10—VARIATION OF TEMPERATURE IN VERTICAL SECTION OF MASS CONCRETE

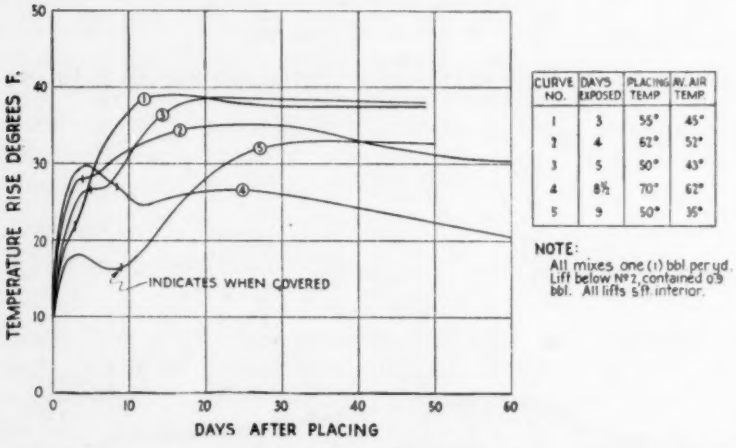

TEMPERATURE RISE DEGREES F.
0 10 20 30 40 50
DAYS AFTER PLACING
INDICATES WHEN COVERED
① ② ③ ④ ⑤
CURVE NO. | DAYS EXPOSED | PLACING TEMP | AV. AIR TEMP
1 | 3 | 55° | 45°
2 | 4 | 62° | 52°
3 | 5 | 50° | 43°
4 | 8½ | 70° | 62°
5 | 9 | 50° | 35°
NOTE:
All mixes one (1) bbl per yd.
Lift below No 2, contained 0.9 bbl. All lifts 5 ft. interior.

FIG. 11—TEMPERATURE RECORDS OF MASS CONCRETE SHOWING EFFECT OF VARIABLES

the same it is seen that temperature rise of curve 5 is about 3° F. more than curve 4. This is the effect of the higher air and placing temperatures for curve 4. The higher temperatures cause the cement to generate heat at a more rapid rate thus permitting more heat to be dissipated during the period of exposure. Comparing curves 3 and 5 for which the placing temperatures were the same the longer period of exposure and lower air temperature for curve 5 caused a 5° F. less

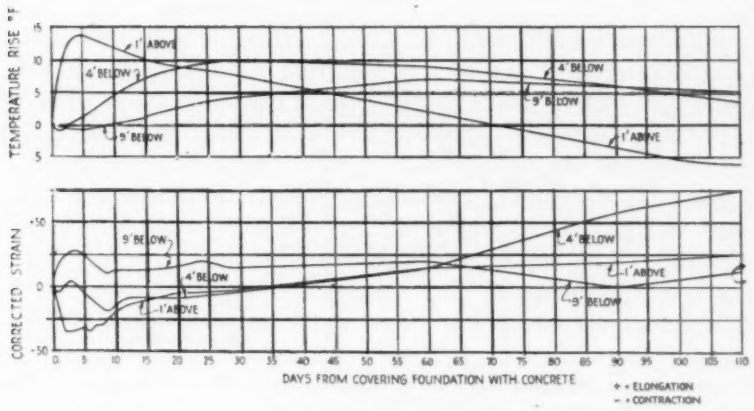


FIG. 12—HORIZONTAL STRAIN METER RECORDS IN ROCK UNDER BLOCK 10

temperature rise. The average temperature rise of all concrete in which thermometers were placed was about 35° F.

Strainmeter Records

The observed reading of horizontal strainmeters embedded in the foundation and concrete of block 10 are shown in Fig. 12. The corrected strains and temperatures are averages for the two groups of meters located at the axis and 94 ft. downstream from the axis. The records of the meters 20 ft. upstream from the axis are effected by the cut-off trench and are not comparable with the other two groups. Corrected strain is obtained by subtracting algebraically the strain caused by temperature change from the actual strain as determined by the meter. If it be imagined that the base of the dam were free to move, the strain measured by the meters would be equal to the strain computed from the measured temperature change and coefficient of expansion and the corrected strain for this case would be equal to zero. The actual conditions of restraint change the results and make corrected strain a measure of stress and because of this fact it is utilized as a check on computed stress under similar conditions. It is believed that a change in moisture content of the concrete at the base of the dam is impossible, therefore all strain is assumed to be the effect of temperature change and stress. Referring to Fig. 12, it is seen that at the age of 3 days the expansion of the concrete that would occur if unrestrained was prevented by the underlying rock causing a corrected strain of contraction in the rock and concrete where the temperature increased and corrected strain of elongation in the under-lying rock where there was no increase in temperature. The temperature of the rock four feet below the base began to rise

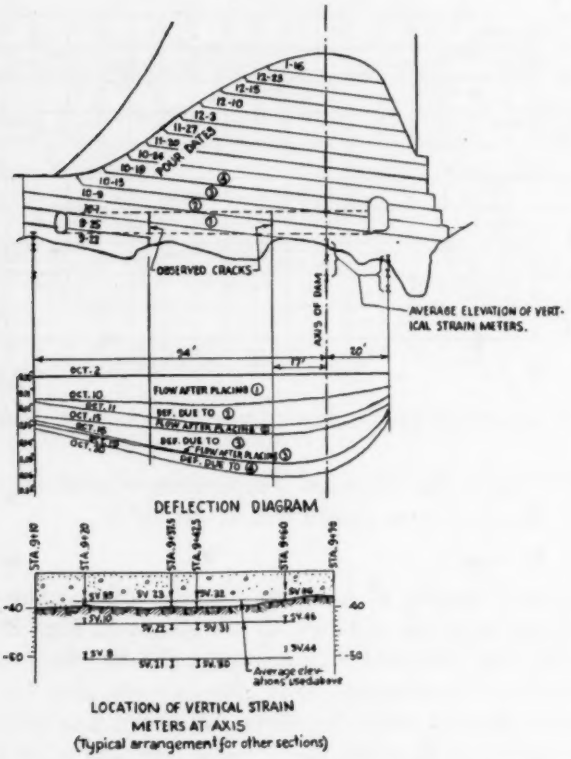


FIG. 13—VERTICAL DEFLECTION OF FOUNDATION

after three days, and the restraint of expansion by the underlying rock caused a corrected strain of contraction. As the temperature of the concrete one foot above the base began to fall the contraction strain decreased and the elongation strain of the rock 9 feet below the base also decreased. These strains were not used to determine stress because of uncertainty as to the plastic and elastic properties of the rock. However a comparison of these strains with computed strains will be made in connection with computed temperature stresses.

The vertical strainmeters embedded in the foundation and concrete of block 10 were located so as to determine the distribution of the heavy concentrated pier load over the foundation. The piers as yet have not been loaded with gates and horizontal water thrust so the chief value of these meters has not yet been realized. However, the meters have given an indication of the distribution of the weight of concrete in a transverse direction and the resulting deflection of the foundation. Fig. 13 shows the sequence of lifts in block 10 and

the deflection curves in a transverse direction. The deflection caused by the two lifts placed on the 22nd and 25th of September which did not cover the whole foundation have been disregarded and the deflection plotted in Fig. 13 are those which occurred after the 2nd of October. The instruments were read daily. The difference between the first reading made after a lift was placed and the previous reading was regarded as elastic deflection of the foundation. The increase in deflection shown by the change in strain of rock during the period between lifts was taken to be plastic. As deflection was determined at only three points and these were not symmetrically located, it cannot be said that the smooth curves of the figure give the true shape of the deflected foundation. A set of meters located at the center of the base would probably show greater deflections. The effect of these deflections on stresses in the concrete have been calculated, and the computation and results will be briefly discsussed later.

6—STRESS ANALYSIS

Thus far the text of this paper has been primarily concerned with reciting factual data that concern Bonneville problems only. The special consideration given to the cement, the properties of concrete mixes and a little later the quality of the concrete as evidenced by visual inspection can now be accepted as the basis for an intelligent analysis of stress in the mass due to thermal changes.

There is a well known principle of structural mechanics which states that if the two ends of a prism are immovably fixed when under no strain and if the temperature is lowered through a known range, then the body suffers a tension proportional to the fall in temperature so long as the stress remains within the elastic limit. If one end were free to move, then for a rise or fall of one degree a unit of length would change by an amount equal to the coefficient of expansion. Any given length then will change by an amount equal to the length times the coefficient of expansion times the number of degrees fall (or rise). If this contraction is prevented a force will be set up which is the product of the modulus of elasticity, the coefficient of expansion, and the temperature change. From this it is evident that for a given temperature change, stresses in different materials are proportional to the moduli of elasticity and the coefficient of expansion, or in other words, their simple rigidity.

A mass concrete dam is usually built up in layers with an interval of time between lifts. The amount and rate of temperature rise in each lift is a function of the temperature of the concrete when placed, the cement content, the air temperature, the depth of the lift and the time interval between the layers. The temperature at any time in the

concrete varies in a manner similar to variation shown in Fig. 10. For the first few weeks after the concrete is placed the variation in temperature is large; at later ages this variation becomes much smaller. Stresses are developed in the concrete by these temperature variations as partial restraint against movement is provided by the foundation, which usually changes in temperature much less than the concrete, and by the concrete itself. The sections which are at the highest temperature being partially restrained against expansion by the cooler sections which are in turn expanded somewhat by the warmer concrete.

It is generally known that the modulus of elasticity of concrete varies with stress and with age. The different ages of the lifts and the variations in temperature with depth in each lift at early ages makes the determination of temperature stresses in mass concrete a much more difficult problem than the example of the prism given above.

An analysis of stresses resulting from thermal changes has been made of one block of the spillway dam in which the above principles have been applied. Block 10 was selected because in it was placed an extensive system of thermometers and in the rock below its base strainmeters were embedded in such a way as to measure both the horizontal and vertical deformations. In addition to this, it is the only block of the first step construction in which cracks were observed to such an extent as to justify consideration from the standpoint of analysis. The value of this type of calculation is that it is not based on a wide variety of assumptions, but on actual measured coefficients of the materials involved. Again, sections for analysis are selected in which instruments have been placed, the results of which are a check on computations.

At the time cracks occurred in block 10, the fall in temperature was only 4°, although it is usually considered that under complete restraint concrete can stand a temperature drop of at least 15°. Therefore, the existing condition must have been more severe than that of complete restraint. Upon examination it was evident that there were other causes contributing to the formation of the cracks that occurred. They may be enumerated as follows:

1. Simultaneous rise in temperature of overlying and underlying material.

2. Effect of vertical load on yielding foundation, resulting in a non-uniform depression of base, the central portion being depressed more than the sides.

3. Rapid application of force due to expanding concrete above.

4. Presence of abrupt changes in cross-section produced by sawteeth in the foundation rock and bulkheads in concrete.

To evaluate the foregoing causes, a step-by-step analysis was made to determine the temperature stresses when the cracks occurred. This analysis takes account of the changing properties of the concrete and is based upon the following fundamental conception of the stress conditions: That shearing stress due to temperature deformation in a block 135 feet long is zero in the central portion of the 135-ft. dimension.

With regard to this statement consider a lift of concrete 135 ft. long placed upon bedrock. The temperature of the concrete three days after placing is highest near the center of the lift; at the upper surface the temperature is the same as that of the air; and at the contact with the rock the temperature is less than the center temperature. At the ends of the lift the normal stresses, or tensile and compressive stresses, resulting from this temperature variation must be equal to zero, and the shearing stresses at their maximum value. Progressing towards the center of the 135 ft. dimension the normal stresses increase and the shearing stresses decrease until the shearing stresses become zero and the normal stresses maximum. Between this point and the corresponding point near the other end the shear is zero and normal forces do not change. The normal stresses are the result of partial restraint of temperature deformation. The portion of the temperature deformation which is not restrained must be equal in amount at all points in a vertical section through the central portion of the concrete. If this were not true a vertical section before temperature change would be warped or rotated after temperature change and the shear would not be equal to zero. The analysis does not take into account the effect of horizontal deformations and stresses parallel to the axis of the dam or the stresses transverse to the axis.

To compute the temperatures and stresses in concrete, the thermal and elastic characteristics of the concrete must be known. The investigations previously described determined the following:

Diffusivity of concrete	0.7 ft.^{2} per day.
Rate of heat generation	Fig. 4
Modulus of elasticity of the foundation rock	400,000 p.s.i.
Plastic properties of the concrete	Fig. 5
Stress strain relations of the concrete	Fig. 6
Coefficient of thermal expansion of concrete	0.000004 in. per in. the first day. 0.000005 in. per in. after the first day.
Coefficient of thermal expansion of the rock	0.000005 in. per in.

The modulus of elasticity of the rock as determined by compression tests under complete lateral restraint on the three types of rock predominating in the first 15 ft. below the concrete gave values ranging from 100,000 to 800,000 p.s.i. An average value of 400,000 p.s.i. was used in the computations. The diffusivity of the rock was found by test on one sample of a type not occurring under block 10 to be less than that of the concrete. Computations, made from the measured temperature records, of the value of the diffusivity gave varying results ranging above and below the value of the diffusivity of the concrete. For this reason the temperature computations were based upon the assumption that the diffusivity of the rock is the same as the concrete.

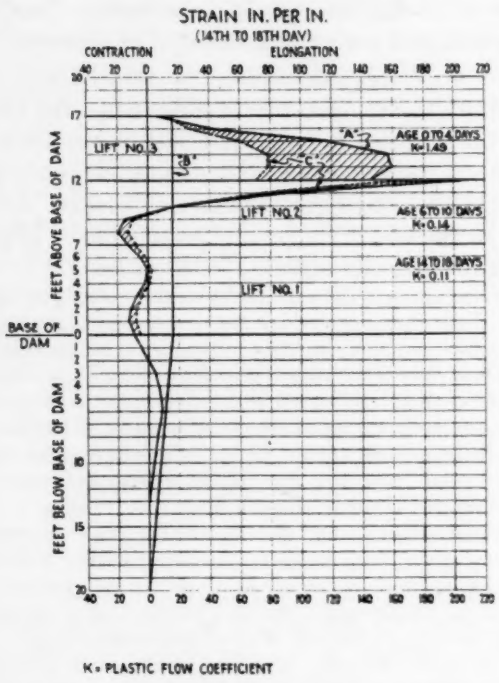


FIG. 14—TYPICAL STRAIN DIAGRAM

The plastic flow coefficient curve Fig. 5 was determined from compression tests on the assumption that flow is proportional to stress. For the computations of stress the additional assumption was made that flow in compression and tension are the same, as test data concerning the plastic properties of the concrete under tensile stress at early ages were not available.

Method of Computation

The temperature in the rock and concrete was computed for each foot of elevation above and below the contact of the concrete with the rock. The intervals of time for which the temperature distribution were computed included the first day after each lift was placed, and remainder of the interval until the placing of the next lift was divided into two or three periods. The cement content, the placing and air temperatures, and the depth and time of placement were taken from concrete inspection reports for block 10. As shown on Fig. 7, the first lift to extend from the contraction joint, 97 ft. downstream from the dam axis, to the upstream heel was placed to elevation —33.0 at the axis. It was this lift that was considered to be the lift in contact with the foundation for the temperature and stress computations which were for a vertical section at the axis of the dam where the first lift was 7 ft. thick.

The temperature distribution having been computed at a vertical plane through the lift for each period the corresponding stresses were computed as follows:

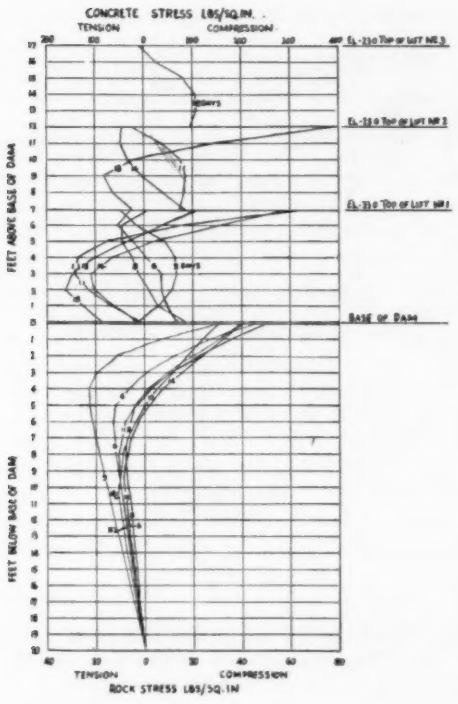


FIG. 15—STRESS DIAGRAM

1. Retering to Fig. 14, line "A" was plotted and represents the unrestrained deformation as determined by the temperature change and the appropriate coefficient of expansion.

2. Line "B" is so located that the summation of stresses resulting from the deformation which does not take place (the difference between the abscissae of the "A" and "B" lines) is equal to zero.

3. All of the restrained deformation does not produce stress because of the plastic nature of the concrete. The shaded area between the "A" and "C" lines represents plastic strain, and the unshaded area between represents elastic strain which produces stress.

4. From diagrams similar to Fig. 14 stresses were computed for the several periods. These values were then combined with stresses present at the beginning of the period and which were adjusted for the plastic relief occurring during the period.

This procedure was continued at adopted time intervals until the age of 18 days for the first lift was reached. At about this time cracks occurred in block 10. The computed stresses are shown in Fig. 15.

The discontinuity in the strain diagram Fig. 14 at point 12 is the effect of 9° difference between the air temperature and the placing temperature of the third litt. The surface of the second lift was at air temperature at the beginning of the period and the placing temperature of the third lift was 9° higher than the air tem-

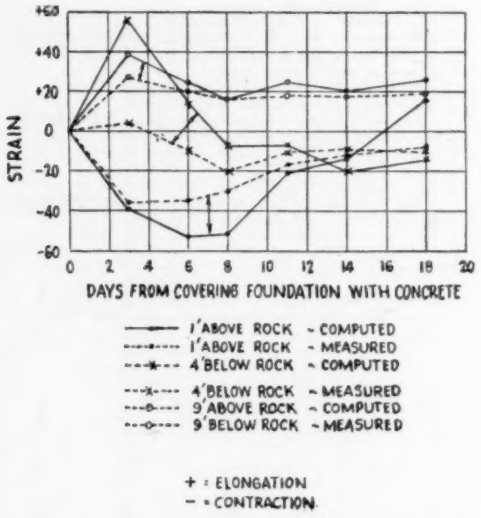


FIG. 16—COMPARISON OF MEASURED AND COMPUTED STRAINS IN FOUNDATION ROCK

perature. The discontinuity in the stress distribution at the junction of lifts was due to this difference in temperature and to the difference in elastic and plastic properties of concrete of different ages.

In Fig. 16 a comparison is made of the computed and measured corrected strains in the rock and in the concrete one foot above the base. The close agreement of the two groups of curves is a convincing check.

Results of Thermal Stress Computation

The computed stresses due to thermal change plotted in Fig. 15 show clearly the discontinuity in stress at the junction between lifts. The similarity of the stress conditions in the second lift on the eleventh day and in the third lift on the 18th day indicates that the stress distribution is typical for an interior lift during the period that temperature is rising and the lift is exposed. The deformation due to temperature is partially restrained by the older concrete below, which does not rise appreciably in temperature. The temperature of the new concrete at the base of the lift does not rise as much as that in the middle of the lift and therefore the stress is less. Also on the eighth and fourteenth days in lifts one and two, respectively, the stress condition appears to be typical for a lift in which the temperature is falling and which is still exposed. Here the loss in heat is through the exposed top surface producing tensile stress in the upper section of the lift. The maximum tensile stresses in the first lift from temperature change alone occurred on the eighteenth day with 150 p.s.i. at two feet above the rock.

Stresses Caused by Vertical Deflection

The vertical deflections of the foundation as shown in Fig. 13 were investigated to determine the stress developed in the concrete as a result of these deflections. The concrete was considered as a beam subjected to a uniform load sufficient to produce the measured deflections. The different ages, and elastic and plastic proper-

ties of the lifts were considered in computing the stress. The results of this computation gave a tensile stress of 50 p.s.i. at the base of the dam at the time cracking occurred which was during the period of exposure of the lift No. 3, Fig. 13. This stress when added to the computed thermal stress gives a total of 200 p.s.i.

Effect of Shape of Foundation

This tension at the bottom developed during a period of about three weeks. Cracks occurred at locations where longitudinal bulkhead joints were placed at the top of ridges left in foundation rock. The two together formed a break in the cross-section of about ten feet in height. It is well known that abrupt changes in section cause a great increase in the intensity of existing stresses. Photo-elastic tests made on models of this particular section indicated the following stress factors in comparison with a perfectly flat base: for sawteeth alone the stress above the sawtooth was 1.6 times the stress for the flat base; for sawteeth plus a vertical bulkhead the local stress above the bulkhead was considerably greater than twice the value where no discontinuity existed; and the case where the base was flat but left rough there was no apparent increased stress. The presence of bulkheads in the first lift caused such severe conditions that cracks would inevitably have occurred even for stresses of small magnitude. Applying a concentration factor of 2.0 to the stress of 200 p.s.i., a stress of 400 p.s.i. is obtained which is more than sufficient to crack concrete three weeks old.

Stresses from Final Cooling

The changes in stress resulting from the placement of additional lifts in block 10 have not been computed, as the object of the computations was only to determine the stress existing at the time the cracks occurred. However, the conditions producing stress are evident and an estimate has been made of the stress when cooling is complete. The remainder of the lifts will have little effect on the tensile stress in the foundation lift as the depth of concrete is great and only a small increase in unit tensile stress balances the compressive stress developed in the new lift. The chief increase in tensile stress at the foundation is caused by the gradual cooling to the final temperature. From Fig. 9 it is seen that in 10 months the base of block at the axis has cooled 20° or at an average rate of 2° per month. The final temperature of the dam will probably be 55°. The temperature of the base of block 10 on September 1, 1936 was 65° leaving 10° of further cooling which would proceed at a slower rate than the previous cooling, probably at about 1° per month.

The extensibility tests of the first investigation showed that under periodically increasing tensile load for concrete made from portland-puzzolan cement the sustained modulus of elasticity was 2,000,000 p.s.i. at the age of 5 months. Under complete restraint, the 20° drop in temperature by September 1, 1936 with a modulus of elasticity of 2,000,000 p.s.i. and a coefficient of thermal contraction of 0.000005 in. per in. per degree, a stress of 200 p.s.i. would be developed. However, complete restraint does not exist and the stress would be somewhat less. This stress combined with that resulting from the final 10° cooling will probably closely approach the 200 p.s.i. mentioned, making a total of 400 p.s.i.

If the effect of stress concentrations due to irregular sawteeth and bulkhead forms be taken into account it would as previously shown multiply existing stresses by at least two, resulting in a possible tensile stress of 800 p.s.i.

FIG. 17—PLACING MASS CONCRETE 8 YARD BUCKETS BOTTOM CONTROL
FIG. 18—THE NORTH FACE OF BLOCK NO. 9

7—CONSTRUCTION EXPERIENCE AND CONDITION OF FINISHED CONCRETE

Following is a brief statement of the problems encountered in both seasons' work and the present (Dec. 1936) condition of finished concrete.

First Season's Experience

Rapid progress was made in preparing bed rock the first season (1935) and the placing of concrete therefore began somewhat earlier

than anticipated which resulted in placing temperatures much higher than planned. For the first and second seasons the placing temperature at the former was about 70° until mid-October when it dropped to about 50° and remained there for most of the mass concrete. During the warm weather of 1935 a false set occurred soon after placement but not until the mass had been vibrated. This early hardening is characteristic of fine cements and was noticeable here because of the contrast with other parts of the project where coarser cements were used. Other than this the workability and lack of segregation of the concrete was very satisfactory. Tests made at Berkeley showed that portland-puzzolan cement stiffened no more rapidly than did normal portland cement produced at the same mill when of the same fineness. Construction records of other projects where fine cements were used disclose the same results. This condition at Bonneville so far as is known to date is accompanied by no ill effects on the physical properties of the concrete.

The higher starting temperature was accompanied by a less total rise, thus resulting in a compensating effect.

Design Changes

In the interval between the first and second step construction, i. e., the summer of 1936, studies were made with a view of improving the design of the second half of the dam, based on knowledge gained from concrete and foundation rock. It was decided to adopt a cement factor of .9 bbl. per cu. yd. of concrete for the mass, and to reduce the slump to a maximum of 2 in. As evidenced by photoelastic studies there can be no doubt but that the sawteeth ledges in foundation rock cause a very considerable increase in existing stresses. Since it was determined that the foundation rock possesses a wide margin of safety in shear, there was no longer reason for continuing to build these sharp breaks as has been done for the first stop. The new design, therefore, called for a reasonably flat base but always with a slight slope downward in an upstream direction. The surface of the rock breaks very rough and was therefore left in this condition to receive the concrete.

Second Season's Work

The second season's work progressed more rapidly than the first and there was no problem with the early set as described above. The concrete was entirely satisfactory from the standpoint of workability, water gain, etc. It is not known just why the early set should vary with cement of substantially the same composition.

The first season's work was with a new concrete plant and new organization; the concrete was conveyed to the opposite end of the

dam, whereas the second years work was a much shorter distance and with a well perfected organization.

Condition of Finished Concrete

Due to the shape of the spillway block and to the special cement used, great interest existed as to the nature and appearance of the concrete as evidenced by surface inspection. The spillway dam has been inspected by numerous visiting engineers, who are primarily interested in concrete, and the unanimous verdict has been that the structure is surprisingly free of cracks, sand streaks, honeycomb and other evidence of segregation. However, on close inspection it is seen that small cracks have occurred in localized areas.

Block 10 was the first built and preceded adjoining blocks by a few lifts. A transverse inspection tunnel in this block disclosed two vertical cracks parallel with the axis and at points distant from the axis corresponding with the tops of ledges or "saw-teeth" as noted on Fig. 1. At these positions bulkhead forms were inadvertently built to the height of about 4 ft. The cracks evidently are extensions of these construction joints. As block 9 was built up its north face was exposed inside the cofferdam and was closely watched. It also soon cracked in positions corresponding to 10. In each case the cracks occurred about 15 days after the lifts containing cracks were placed. Brass plugs were installed in such a way as to measure the changing width. No other cracks of any kind were observed during the working season of 1935-1936 although a thorough inspection was made. Not only were no other cracks visible but the surface was also largely free of crazing, honeycomb, sand streaks and lime deposits.

As soon as the water subsided in August 1936, the interior of all blocks were again examined, and cracks known to exist in block 10 were measured and found to have closed somewhat. Those on the north face of block 9 were also measured and found to be about the same as the season before. In November 1936 a crack survey was made in which all finished surfaces were closely inspected and a record made of all imperfections then visible. The second half of the dam was then substantially finished up to the spillway crest and the survey included surfaces visible on all faces of blocks, piers and inspection tunnels as well as many large exposures in wing walls and abutment structures. As a part of this survey, the actual width of all cracks was measured by the use of a Brinell microscope in which the readings are direct to 1/10 of a millimeter. It was also found entirely practicable to further estimate to 1/100 of a millimeter which permitted an accuracy of 0.00039 in. The structural cracks in block 10 were found to be 0.027 in. in width at this time. Since the adjoining

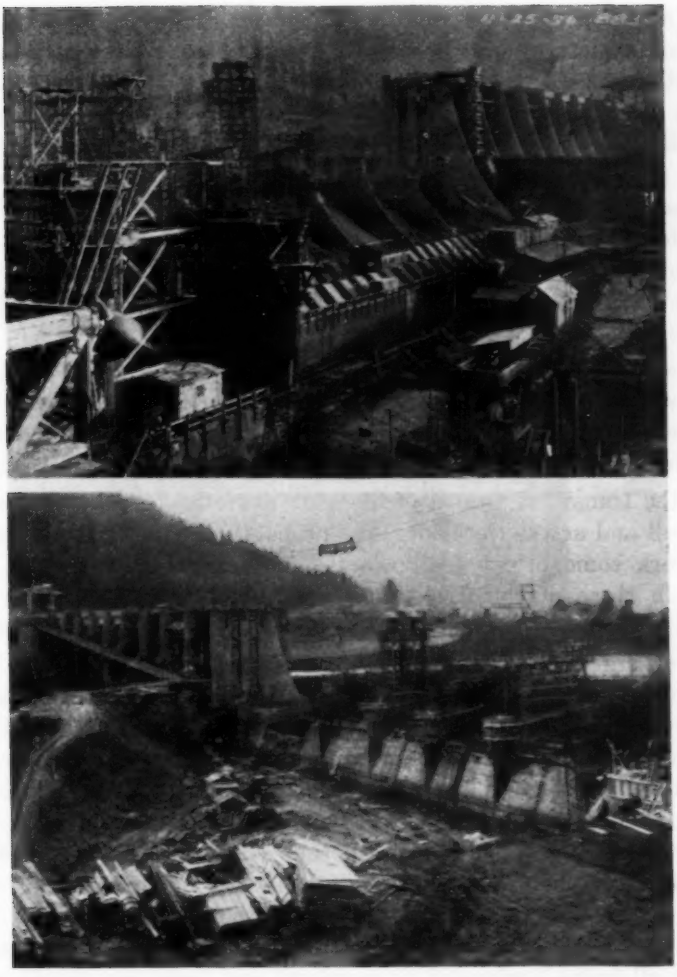


FIG. 19—LOOKING UPSTREAM—2ND STEP CONSTRUCTION IN THE FOREGROUND
FIG. 20—LOOKING DOWNSTREAM—2ND STEP CONSTRUCTION IN THE FOREGROUND

block on the north of block 9 covered the cracks there, it was impossible to measure them. Some leakage occurs in both cracks in block 10 and a small amount of lime deposit in the surface indicates that leaching will take place even though the amount of water present is very small. At one other point only is this surface deposit visible and that is the upstream face of main inspection tunnel where the wall is damp and discolored for a distance of about ten feet.

In addition to those mentioned, the crack survey disclosed three vertical structural cracks over the entire barrel of the transverse tunnel of block 5; the widths being respectively 0.005, 0.005, 0.015 in. There are also numerous others in block 5 both vertically and horizontally but disconnected. There was an almost uniform width of 0.004 in., and at intervals of from 3 ft. to 10 ft. On the spillway crest of this block two diagonal cracks were evident which on examination were found to be 0.005 in. wide, which were entirely closed and when seen under the microscope appeared to be tightly filled with material cemented in place,—thus perhaps falling in the class of self-healing cracks.

Attention should be called to the fact that block 5 was built much in advance of adjoining blocks for a large part of its mass. This resulted in rapid cooling and was possibly responsible for the excessive cracking which occurred in this block as compared with all others.

In the transverse tunnel of block 18 a vertical crack was found in one wall and across the floor, thus probably extending to the foundation rock some fifteen feet below. This crack was 0.012 in. wide. Thus in three of the five transverse tunnels vertical cracks occur and in the two others none occur that are visible under close inspection.

Other cracks were evident in the tunnel walls in limited areas. They were found to be about 0.005 in. wide and open. It was thought, however, that they are in the surface only since those passing through bolt holes with smooth interiors appeared to terminate at about 1½ in. under the surface. They are all disconnected and may have resulted from the cold waters of the previous seasonal flood a few weeks earlier. Study of the isothermic charts indicates a maximum drop of about 20° F. in temperature below the maximum.

The crack survey disclosed two other conditions, both of which appeared on the piers. In numerous places cracks were recorded almost all being horizontal or nearly so and from two to ten feet in length. Not all were measured but those that appeared to be the most extensive were measured and found to be 0.004 to 0.005 in. wide and closed in much the same way as described on the surface of block 5; which leads to a possible conclusion that a self-healing crack may be one that occurs in a zone of stress. So called autogenous healing is doubtless assisted by subsequent hydration of deposited material. In both cases mentioned, the stress is the dead load of the structure only.

Other signs of cracking on piers, until more carefully examined, had been called crazing. Isolated areas had a spiderweb appearance, which on close examination was found to be in a thin film on the sur-

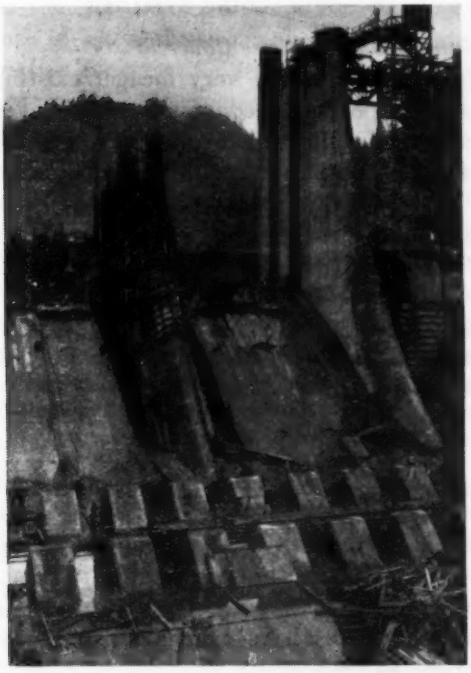


FIG. 21—LOOKING UPSTREAM—2ND STEP CONSTRUCTION

face only. When ground down and studied under the microscope the surface was entirely unbroken. This film undoubtedly collects on the smooth surface of the forms and at some stage of the process shrinks or weathers enough to produce the rather unusual effect. At numerous other places, throughout the period of construction, reinforced concrete structures exposed for a period of months, were closely inspected and disclosed no cracks with the exception of one wing wall which exhibits three small cracks similar to those on piers. These structures were generally reinforced with about 0.4 per cent temperature reinforcement and were often from 75 ft. to 100 ft. long without joints. The cement factor in these cases was about 1.4 bbl. per yd. of concrete and curing was 14 days with cold river water. The superstructure consisting of extremely heavy reinforced concrete girders, parapet walls, floor panels, etc. is free of cracks and other surface defects.

From a study of these data it can readily be seen that the cracks in the structure range from 0.027 in. downward—which appears to be an unusually satisfactory condition.

8—CONCLUDING REMARKS

Portland-Puzzolan Cement in Mass Concrete

It is recognized that with the very meager existing knowledge of elastic and plastic properties of concrete at early ages and the little known influence of restraints which ordinarily exist in mass concrete dams any conclusions at this time must be regarded as not fully substantiated. However, a single case such as the Bonneville concrete carried through to its logical sequences may be of some value.

The effect of the cement alone is small but it cannot fairly be considered alone. An incomplete list of these conditions which may be said to influence the use of cement in mass structure is as follows:

Placing temperature of concrete, final stable temperature, heat of hydration of cement, cement factor, character of aggregate, resulting tensile and compressive strength of concrete, elastic and plastic properties of concrete, dimension of sections as determined by design; artificial or natural methods of cooling, size and thickness of lifts, time of exposure of each lift before being covered, uniform rate of construction with relation to adjoining lifts, foundation rock including its elastic and plastic properties and the shape and preparation of the contact surface.

Portland-puzzolan cement as used at Bonneville has given highly satisfactory results. It seems safe to say that it deserves an increasingly important place among special cements and when used in carefully designed mixes the resulting concrete may be said to have the following advantageous properties:

1. Improved workability.
2. Freedom from segregation and water gain.
3. High factor of impermeability as compared with low heat portland cement.
4. In comparison with modified portland cement a greater percentage of total heat of hydration is developed at early ages and less total heat of hydration.
5. Relatively high early tensile and compressive strength coupled with high early and continued high plastic properties.
6. Long continued gain in strength.

During the design period the effect of the above mentioned properties should be carefully reconciled with other conditions influencing the choice of cement for mass concrete. The result will make possible a decision as to whether or not portland-puzzolan cement should be used.

A Construction Program to Minimize Tensile Stresses

A construction program can be adopted before contract commitments are made which will minimize the stresses due to thermal changes as a part of the program of construction which can readily be set up in advance. The cement properties or better than that definite control properties for concrete can be specified. The engineer has only to recall the rapid strides made in recent years in concrete manufacturing processes beginning with the promulgation of the water

cement ratio and the Abrams fineness modulus curves for aggregates to have confidence in his ability to make much greater strides than in the past. In the construction industry the engineer must take the lead with only professional progress as his reward. There has been a full measure of this in the past, but anyone who works with concrete knows that the ultimate standards of construction are not yet known. Again to quote from J. L. Savage and others in their most excellent paper on "Special Cement for Mass Concrete": "It might be said that the ideal concrete dam would be one involving sufficiently low volume changes to permit its being built as a monolith without fear of cracking, and of concrete which would withstand indefinitely the rigors of severe climatic or other disintegrating conditions."

An improved program of construction would not entail revolutionary changes in present methods. A suggested outline might well include the following:

1. Specify a concrete (or cement) which will meet definite requirements as to heat of hydration, diffusivity, permeability, strength in both tension and compression, plastic flow properties within an allowable range when tested under construction conditions.
2. Specify the permissible heat range in various portions of the mass when considered with time. This should necessarily take into account the placing temperature and final stable temperature to be expected.
3. The rate of construction should be so controlled that the requirements in 2 could be met and so that too rapid building of one block as against other portions of the structure would be prohibited. This is emphasized by the results of block 5 at Bonneville. It is entirely possible that a graduated time schedule could be followed with profit.
4. There should be some latitude in construction requirements depending on what is disclosed by bed rock conditions when exposed. This is especially important if a variable condition exists as at Bonneville.

The above would not upset the competitive phase of contract awards so long as all bidders are on the same basis and should not result in greatly increased costs if the builders have full knowledge of conditions to be encountered before planning their organization.

A Method of Design Analysis Taking into Account Tensile Stresses

It is recommended that a method of stress analysis in the design of mass concrete dams be adopted in which the effect of thermal stresses is taken into consideration as well as stresses due to gravity and water loads. The usual method of mass concrete design as applied to dams does not take this important principal into account. Principal stress studies and foundation loading assume that the entire section or a well defined part thereof acts as a monolith. The redistribution of stresses due to cracks is not known. In dealing with cracks in gravity dams, while they are generally admitted to be objectionable,

they are sometimes thought of as a construction joint built by nature where none was otherwise provided. It has been stated on good authority that cracks of less than 0.01 in. in width are surface cracks only. A general division of cracks into surface and structural cracks may be made in which the latter includes all cracks that are extensive, that are well defined in width, are of sufficient length and in such position that if open would divide the mass section and cause stresses to be redistributed from that computed for the intact section. A vertical crack parallel with the axis of a gravity dam if extensive certainly falls in this class of structural cracks. The only way it can develop shearing strength is through displacement of one section. Deposit in the crack and subsequent hydration of deposited materials may later heal it. On the other hand weathering and seasonal temperature fluctuations may through a long period of years result in a condition that causes failure and calls for repairs. Early concrete pavement on American highways is an extreme example of this. In any event it is herein assumed that structural cracks in mass concrete dams are objectionable, and if reasonably possible, design and construction should be so conducted that they will not occur. The fact that at Bonneville for instance, cracks formed in some blocks and did not in others, indicates that conditions may be close to a state of equilibrium and if a more exact measure of thermal stresses were known greater safety and longer life could perhaps be attained by slight changes in an otherwise well balanced design.

It has been pointed out that a crack occurred when only a 4° drop had taken place but was assisted by expanding rock below and concrete above. An analysis should therefore set up the relative changes that can be allowed between the lifts of predetermined thickness rather than the allowable drop for a single lift. It can readily be shown that a rather large drop can be withstood by a single lift if conditions are favorable. The first lift should be placed on a prepared base that is rough but free from large and abrupt changes in cross section and care exercised that no vertical joints are allowed which would thus constitute the beginning of a vertical crack. It is possible that due to its close contact with the rock its power to resist stresses would probably more nearly approach that of the modulus of rupture of concrete instead of pure tension. The following summary is in support of this method of design.

1. Uncertainty exists as to the effect of cracks on the distribution of gravity loads.

2. It is possible to determine the degree of safety against structural cracks in any definite plan of construction.

3. To ignore tensile stress studies may result in a design in which the factor of safety in this respect is less than 1, whereas in all other respects ample safety exists.

4. It is therefore recommended that temperature-tensile stresses be taken into account. To do so follows well established practice in engineering design of other structures in which such stresses exist.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the valuable assistance of R. E. Davis in supplying data on cement investigation, cement specifications and on trial mix tests. To Roy W. Carlson as consultant on interpretation of results obtained from embedded instruments and conclusions therefrom. To H. D. Eberhart for assistance in computations and to I. E. Burks in supplying data on trial mix tests, routine testing and description of job experience.

Design and construction of the Bonneville Project is under the direction of the Corps of Engineers, U. S. Army, Col. T. M. Robins, Division Engineer; Lieut. Col. C. F. Williams, District Engineer; Capt. Colby M. Myers, Resident Engineer. Civilian engineers connected with the work are C. I. Grimm, Head Engineer; A. E. McKennett, Principal Engineer; R. R. Clark, Designing Engineer. In the field B. E. Torpen, Senior Engineer; O. C. Hartman, Resident Engineer of the Spillway Dam; I. E. Burks, Chief Concrete Technician.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by April 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for May-June, 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MAY-JUNE 1937

Discussion of a paper by Clark and Brown:

Portland-Puzzolan Cement as Used in the Bonneville Spillway Dam*

CONVENTION DISCUSSION

Raymond E. Davis:† Mr. Clark has mentioned the design peculiarities of the dam at Bonneville, it is a relatively low, thick structure and for that reason is more likely to crack than would a dam of normal proportions. Conditions surrounding the site at Bonneville include a relatively yielding foundation, a moderate climate where frost conditions are not severe, but where, during the winter months, the temperatures are frequently as low as 20° F. and sometimes as low as 10° F.; where in November, December, January and February, nights frequently produce freezing temperatures with temperatures above freezing during the day.

Between piers 10 ft. thick, at intervals of sixty feet, will go gates to control the water which flows over the spillway. Normally this portion of the dam after construction has been completed, will be under water. A much richer concrete was employed in the piers than for the mass of the dam. During the first construction season, no scaling or spalling whatever of any of the concrete was observed, even though in the first construction season we had some weather colder than we have had so far this year. This season we have observed some surface scaling through the frost action. During the month of October the average temperature was 53° F. There were no frosts of consequence during October. In November the average temperature to which that concrete was subjected was 45 degrees and there were a considerable number of nights as there were also during the months of December and January, when there was freezing with thawing during

*JOURNAL Amer. Concrete Inst., Jan.-Feb. 1937, *Proceedings* Vol. 33, p. 183. Presented by R. R. Clark at 33rd Annual Convention, New York, Feb. 23-26, 1937.

†Professor of Civil Engineering, University of California, Berkeley.

the day. At the age of 14 days portions of the dam previously subjected to an average air temperature of 15 degrees, were used as walkways and traffic from one end of the dam to the other was severe. Under this traffic there was noted some abrasion of the surface of the weak concrete. Later, as the section was built up, water was continually or intermittently trickling down from the curing of the concrete above. Under those conditions there occurred the frost action of which I speak, which produced surface scaling. This illustrates first, that concrete at a temperature of 50° or lower hydrates slowly and acquires strength slowly, and second, that where such concretes are continually subjected to the action of freezing and thawing, even though the temperature does not go very much below freezing, surface scaling will take place. I believe that this concrete hardened less rapidly than would normal portland cement concrete. It is therefore particularly necessary when using portland-puzzolan cement, that there be adequate protection against low temperature conditions. There was no scaling on the richer concrete in the piers. When water of high velocity began to pass over the dam, some of the concrete in the piers was not more than a month old, and during the period of the life of that concrete, the temperature had been in the range of 40° to 50° F., resulting in a surface of low strength. Under these conditions it is fair to assume that erosion of the pier noses of which Mr. Clark has spoken would be somewhat more severe than had the concrete been cured at normal temperature. Any slow hardening cement, whether a portland-puzzolan or a low-heat portland of the Boulder Dam type, doubtless requires a greater degree of winter protection than would a normal portland or a modified portland cement. I had the good fortune last summer to make an extensive tour looking at dams, and I may say that the degree of cracking as evidenced by the number of cracks in the Bonneville dam, as well as in the auxilliary structures that are not of mass proportions, is very much less than in dams of recent construction where the modified low-heat cement has been employed. In this respect the portland-puzzolan cement is on a par with low-heat portland cement. This may be explained by the fact that portland-puzzolan cement concretes yield more readily during the early hardening period than do corresponding concretes employing the modified type of portland cement. This yielding, as I visualize it, relieves stresses that would otherwise be set up in these structures due to thermal changes. Mr. Clark has mentioned the stresses that may exist between a lift below and a new lift above, or the stresses in the outer, cooler surface of the concrete when the center of the mass is at a much higher temperature.

I do not think that every portland-puzzolan cement that may be manufactured is necessarily a good cement for dam construction, but I do believe that properly constituted portland-puzzolan cements possess certain advantages over portland cements for hydraulic structures in general and for mass structures in particular.

D. S. Colburn:* I should like to ask Mr. Clark or Mr. Davis whether there was a change made in the puzzolan requirement to permit the use of uncalcined puzzolan?

Professor Davis: All of the material which has been used as puzzolan has been calcined at a temperature of about 1650° F. and then quickly cooled; none of it has been used in the spillway dam which has not been calcined.

Mr. Colburn: My reason for the question is that at the time the bids were being taken for this cement, if I remember correctly, there was an addenda issued permitting the use of uncalcined puzzolan.

Professor Davis: As far as I recall it was never even suggested, except on the part of cement manufacturers, that we should use uncalcined material.

Mr. Colburn: I think I saw that addenda; I will get a copy of it and send it to you.

Professor Davis: It is some time ago and I might be wrong. I would like to ask Mr. Clark if he recalls any such incident?

Mr. Clark: I recall a conference with the cement people, but I can verify what Professor Davis has said; that all puzzolan material was calcined.

Mr. Colburn: Another thing—was some of this cement made with oil added to it?

Professor Davis: Yes; to eliminate early stiffening observed at the beginning of the constructions of the first section, a very small amount of mineral oil was used by the manufacturer in the process of grinding the cement. The effect of the mineral oil was, as far as we can determine, to reduce the early strength slightly but it made no difference in the later strength.

Mr. Colburn: I understood that it reduced the early strength to some considerable extent and that your contractors were not able to

---

*Marquette Cement Manufacturing Co., Chicago, Ill.

remove their forms very quickly. I further understand that there was cement on the job and some at the mill about which there was some question as to what was going to happen to it.

Professor Davis: At the beginning there was faulty manufacturing control in adding the oil and a few carloads of cement arrived on the job with about four tenths of one per cent of oil instead of a tenth of a per cent. This cement was deficient in early strength and the contractor had some trouble with his forms when that cement got into the job before it was discovered.

Mr. Colburn: What I want to bring out is that many of us might go away with the idea that everything was all rosy as far as the cement was concerned, and I never saw a job yet where it was all rosy. I think we should get a look at the other side too. I want to make two observations: Tests (page 196) appear to be reported as averages. My observations are that tests are worthless reported in averages. We cannot make cement that way. We cannot ship a car that will break at two hundred and fifty pounds and another that will break at three hundred and fifty and say the average is three hundred and therefore it all passes. Another thing, the authors make a positive statement that portland-puzzolan cement as used at Bonneville is giving very satisfactory results. That I can not accept because the job is not yet done.

Chairman P. H. Bates: It happens that I had the pleasure of visiting this dam about three weeks ago and I was also interested in a number of features there which have been adequately covered by the authors. I was particularly pleased to hear Professor Davis call attention to the fact that when you depart from the use of standard cements you must begin to depart from standard practices in their use. If you are going to use special cements, you cannot follow standard practices. That unfortunately was done in that particular condition in which freezing and thawing resulted in disintegration. I might say however, that this freezing and thawing did not occur only with consequent spalling on the deck sections, but there was also evidence of freezing and thawing which was shown to me on an upstream section.

The erosion which Mr. Clark referred to, I think he very properly and adequately explained. There was a condition of a great deal of foreign and abrasive matter and in addition it was whirled and tossed about on certain sections, so it is not at all surprising that, as a consequence, there should be erosion. But there are also some places

on the deck not subjected to that scouring action where there has been erosion which has given some concern. I was told that some tests are being started to determine the relative abrasion of portland-puzzolan concrete and standard concrete. When we are using relatively new materials like this,—that is new in their use in this country, but not in their history,—it is well worth while that we make more complete tests. I believe nobody thought of making an abrasion test on concrete where this type of cement was used, and it is very fortunate that the engineers are now making tests which will give us some information on this subject.

Professor Davis: Abrasion tests were made for portland-puzzolan cement concretes as compared with portland cement concrete before the work at Bonneville was undertaken. Those abrasion tests were made in connection with a series of investigations for the Metropolitan Water District of Southern California, but there was this difference: test slabs, cured continuously at 70° F. were not subjected to abrasion tests until they were about six months old. This concrete at Bonneville was subjected to abrasion when it was perhaps only a month old, cured at 40°-50° F. The tests showed a little in favor of portland-puzzolan cement concrete over portland cement concrete.

R. W. CARLSON*

(BY LETTER)

The authors have presented as one part of their paper a step-by-step method of determining thermal stress (and tendency toward cracking) from computed temperature changes of the concrete. They have taken into account the conditions that prevailed at Bonneville Dam, including the discontinuous progress of casting and the changing elastic and plastic properties of the concrete. This is an important endeavor and could well have been presented in a separate paper.

A method of stress computation, such as that offered, has great value in permitting the prediction of stresses before a mass of concrete is cast, and thus permitting intelligent choices of cement type, of rate of construction, and of spacing of joints. The following remarks are made with an appreciation of the value of the treatise, only hoping to clarify a few minor points.

The method of computing temperature distribution is not given in the paper, even though it is not believed to have been published previously. Since this method, or some other equally good method of

*Associate Professor of Civil Engineering, Mass. Institute of Technology, Cambridge, Mass.

determining temperature changes, is necessary in making the stress analysis, it seems appropriate to state that a description of the method is now being prepared for publication.

The authors have imposed the condition that the summation of horizontal forces on a veritcal section must equal zero, as noted in the second paragraph on page 209. This is not a necessary condition because it demands that there is no net force exerted by the concrete and the foundation as they attempt to expand horizontally. In the typical strain diagram given, however, this makes but little error; the indicated actual expansion of 20 millionths probably should be nearer 10 millionths. Data on the actual degree of restraint that prevails are being obtained in other massive dams, the results seeming to indicate such small variations that the degree of restraint might be assumed without serious error. If these findings are borne out, it will be unnecessary to impose the incorrect assumption that net horizontal stress must be zero.

It is seen in the typical strain diagram, Fig. 14, that the strain is drawn to zero at 20 ft. below the base of the dam. This is evidently an arbitrary choice and probably is consistent with the nature of the foundation. There apparently is no advantage in applying the mathematical theory of elasticity to the foundation because of discontinuities represented by random joints and seams.

The greatest handicap in attempting to make any analysis of thermal stress in mass concrete is the lack of complete information on the elastic and plastic properties of the concrete during early ages. Concretes differ widely in their early properties. These properties are now being investigated in several progressive laboratories and it is believed that they will help in the further development of the method of stress analysis that has been offered by Messrs. Clark and Brown.

AUTHORS' CLOSURE

The authors are grateful to Mr. Carlson for his valued discussion of the stress analysis. It is recognized that this method of stress analysis given is but the beginning of what it is hoped will eventually become an accepted step in the design of mass concrete structures. Other than this, the value of the discussion is the emphasis on the importance of special practices governing the use of special cement. Experience at Bonneville bears this out. Two days' work of adjoining sections present different surface appearance, even though the close control was not in the least relaxed. It indicates that for any mass concrete built under difficulties, the surfaces of both good and poor

areas may be close to the border line of satisfactory results. A new series of scour or abrasion tests of portland-puzzolan cement and standard portland cement concrete in progress at Bonneville for four months indicates a possible slightly greater scour on portland-puzzolan specimens. These specimens were, however, cured at standard temperature and do not indicate the disadvantage of low temperature curing.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY 1937

Earthquakes and Reinforced-Concrete*

By Jacob J. Creskoff†

Violent earthquakes are as old as the earth. They occur frequently, averaging one a week throughout the world.(1) They cause 28,000(2) deaths, untold misery, and millions of dollars of property damage annually.

Earthquakes will probably continue to occur as long as the earth exists. However, if knowledge now available to engineers and architects is applied in future design, major earthquakes, in time, need no longer seriously menace life and property.

In order that the importance of the earthquake problem in this country, and the fundamental principles of aseismic design will be properly appreciated, this discussion will inquire into

(1) the distribution and frequency of destructive earthquakes in the United States
(2) seismic genesis
(3) the nature of the ground vibrations
(4) their effect on structures
(5) earthquake damage
(6) aseismic design

as well as into

(7) the characteristics of reinforced-concrete which make it particularly suitable for use in earthquake-resistant construction.

EARTHQUAKES IN THE UNITED STATES

It is commonly believed that destructive earthquakes occur only in particular regions of this country. Analysis of the earthquake history of the United States discloses, however, that it is a mistake to assume that they are confined to a few localities. Also, a number of

*Received by the Institute Dec. 28, 1936 for presentation at the 33rd Annual Convention.
†Consulting Engineer, Philadelphia.

shocks are found to have occurred in regions where, from geological conditions. great earthquakes would not be expected. Again, sections long dormant have suddenly become active.

All in all, the data are of such nature that it would be rash to predict immunity from earthquake damage for any region.

Earthquakes are rated numerically on the Rossi-Florel Scale of Intensity, according to damage done. Thus intensity 8 describes earthquakes which topple chimneys and crack walls; intensity 9, those which destroy some buildings partially or totally; and intensity 10 those which cause great structural disaster.

Using the Rossi-Forel Scale of Intensity as a guide, it is found[3][4][5] that destructive earthquakes have occurred chronologically in the United States as listed in Table 1.

TABLE 1—DESTRUCTIVE EARTHQUAKES IN THE UNITED STATES

(Aftershocks not included)

Year	Place	Intensity	Year	Place	Intensity	Year	Place	Intensity
1638	Massachusetts	8	1881	California	8	1914	Maine	8
1662	New England	8	1884	Pennsylvania	8	1915	Nevada	9
1663	New England	9	1884	Ohio	8	1915	California	8
1727	Massachusetts	8	1884	New York City	8	1915	Nevada	10
1737	New York	8	1885	California	8	1915	California	9
1744	Massachusetts	8	1886	South Carolina	10	1916	California	8
1755	Massachusetts	10	1887	California	8	1916	North Carolina	8
1769	California	9	1887	Arizona	9	1916	Alabama	8
1790	California	10	1888	Nevada	8	1917	Missouri	8
1791	Connecticut	9	1889	California	8	1917	California	8
1800	California	9	1890	California	9	1918	Virginia	8
1803	California	8	1891	California	9	1918	New Mexico	9
1806	California	8	1892	California	10	1918	Maine	8
1810	New Hampshire	8	1893	California	9	1918	California	9
1811	Missouri	10	1894	California	8	1919	California	8
1812	California	10	1897	Missouri	9	1920	California	9
1812	Missouri	10	1897	California	8	1921	Utah	9
1817	Massachusetts	8	1897	New York	8	1922	California	8
1830	California	8	1897	Virginia	9	1922	California	9
1836	California	10	1898	California	8	1923	California	9
1838	California	10	1899	California	8	1925	California	9
1840	California	9	1900	Utah	9	1925	Texas	8
1840	Pennsylvania	8	1901	California	8	1925	Montana	10
1843	Missouri	8	1901	Utah	9	1925	New Hampshire	8
1852	California	8	1902	California	8	1926	California	8
1853	California	8	1902	Tennessee	8	1927	California	8
1855	California	8	1902	Utah	8	1927	Mississippi Valley	8
1856	California	8	1903	California	8	1928	California	8
1857	California	10	1904	Maine	8	1929	New York	9
1858	California	8	1905	California	8	1929	California	8
1860	California	8	1905	Idaho	8	1930	California	8
1861	California	9	1906	Michigan	8	1931	New York	8
1863	California	8	1906	Arizona	8	1931	Texas	9
1864	California	8	1906	California	10	1931	Ohio	8
1865	California	10	1908	California	8	1931	California	8
1868	California	10	1908	Pennsylvania	8	1932	California	8
1869	California	8	1909	California	9	1932	California	9
1869	Maine	8	1909	Utah	8	1932	Nevada	10
1870	New England	8	1910	California	8	1933	Nevada	8
1871	California	8	1910	Utah	8	1933	California	9
1872	New York	8	1910	Arizona	9	1934	Nevada	9
1872	California	10	1911	California	8	1934	Utah	9
1872	Montana	8	1912	Arizona	8	1934	California	8
1873	California	9	1913	South Carolina	8	1935	Montana	9
1878	Missouri	8	1914	California	8	1936	Oregon	8
1879	Florida	8						

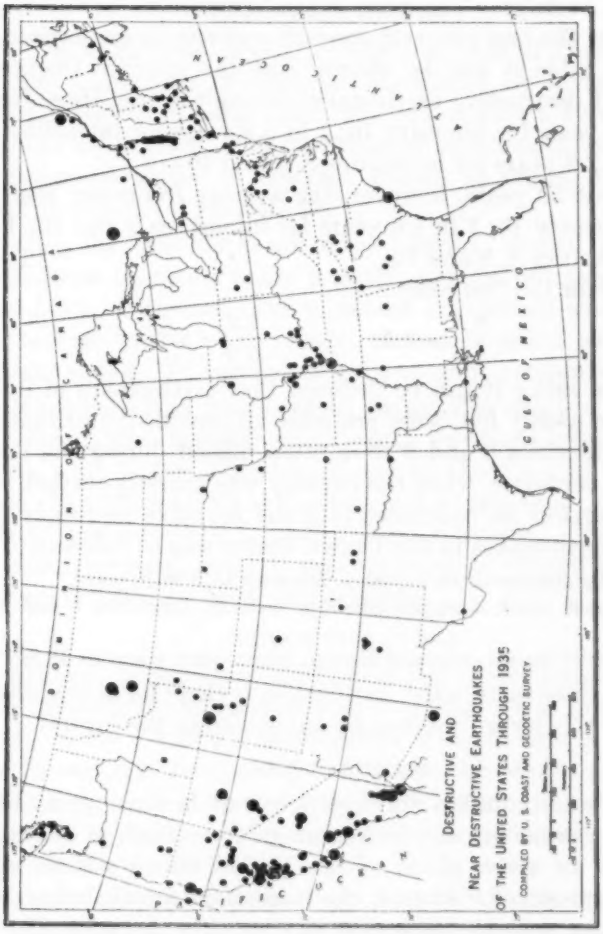

DESTRUCTIVE AND
NEAR DESTRUCTIVE EARTHQUAKES
OF THE UNITED STATES THROUGH 1935
COMPILED BY U. S. COAST AND GEODETIC SURVEY
GULF OF MEXICO
PACIFIC OCEAN

FIG. 1—EARTHQUAKE DISTRIBUTION IN THE UNITED STATES. COURTESY U. S. COAST AND GEODETIC SURVEY

Fig. 1 shows the distribution of destructive and near-destructive earthquakes in this country. It emphasizes that there is hardly any section of the United States which has not experienced seismic shock.

The table lists 136 destructive earthquakes in this country for the past three centuries. Of these 85, or 63 per cent, occurred in the past 50 years. Does this mean that the seismicity of the United States is increasing? Perhaps. The more logical conclusion, however, is that the record for the first two and one-half centuries is incomplete.

From the table it can be shown that the average frequency of occurrence of destructive earthquakes during the past three centuries was: 1 in 10 years for intensity 10; 1 in 5.5 years for intensities 9 and 10; and 1 in 2.2 years for intensities 8, 9 and 10.

For the past 50 years, however, the average frequency was: 1 in 8 years for intensity 10; 1 in 1.6 years for intensities 9 and 10; 1 in 0.6 year for intensities 8, 9 and 10.

The ratios for the two cases are:

$$\frac{10}{8} = 1.3 \qquad \frac{5.5}{1.6} = 3.5 \qquad \frac{2.2}{0.6} = 3.7$$

From these ratios it can be deduced that earthquakes of intensity 10, which are widely felt, were probably all recorded; and that earthquakes of intensities 8 and 9 were often missed during the first two and one-half centuries, when the country was sparsely settled.

It may therefore be concluded that the probable frequency of desdestructive earthquakes in the United States was as follows:

(a) Potentially disasterous earthquakes, intensity 10, 1 in 10 years.

(b) Earthquakes which damaged buildings severely, intensities 9 and 10, 7 in 11 years.

(c) Shocks which caused structural damage, intensities 8, 9 and 10, 18 in 11 years.

In view of these facts, what justification is there for acting under the premise that this record will not be duplicated in the future?

SEISMIC CYCLES

Destructive earthquakes are more frequent in some years than in others. They occur in more or less irregular cycles([6]) of activity. It appears that an active phase of the United States seismic cycle is now being approached. In fact, the indications are that this country will experience several important earthquakes between the years 1939-1946.

The last statement must not be construed as a prediction. Earthquake prediction, to be of value, should be specific as to time, place, and intensity. It should give accurately the time to a day, the place to within a degree of latitude and longitude, as well as the intensity of the destructive earthquake. Thus, the statement that the next

important seismic period in the United States will occur between the years 1939-1946 is not a prediction, but only a probability deduced from statistical analysis of destructive earthquake occurrence in this country during the past three centuries, and from other considerations. As such, it is presented to engineers and architects merely to emphasize the importance of the earthquake problem in the United States.

EARTHQUAKE GENESIS

According to modern theory, the earth consists of a molten core of nickel-iron 4,350 miles in diameter, enveloped by several layers of dense materials 1,750 miles thick, and covered by a 50 mile layer of rock known as its *crust*. The earth's crust is not solid, but is divided by innumerable planes of cleavage known as *faults*, into immense irregular blocks called *fault-blocks*.

Normally, the fault-blocks are in equilibrium. Sometimes, however, owing to forces operating under probably definite but as yet unknown physical laws, contiguous blocks become unbalanced and move with respect to each other. The relative motion is called *slip*, and may be horizontal (Fig. 2), vertical (Fig.3), or some combination of both.

If slip is gradual, it may produce no sensible effect at the surface. If it is abrupt, then invisible elastic earth waves are propagated in all directions. It is the vibration caused by these waves in passing a point on the surface which is known as an *earthquake*.

Earthquakes caused by slip are known as *tectonic*. There are other types of earthquakes, but the vast majority of United States earthquakes are believed to be of tetonic origin.

GROUND VIBRATIONS

As used here, *period* refers to period of vibration, or the time interval between consecutive vibrations; and *amplitude* refers to amplitude of vibration, or the maximum displacement from the axis of equilibrium.

When a region is acted on by impact, or the sudden application or release of a force, it is thrown into *free* vibration. Every locality, if acted on by impact, will vibrate freely in a dominant period, or periods, peculiar to itself.(7)(8)

If, however, a region is acted on by a periodic force, or a force which varies with time, the ensuing vibration will be *forced*, and will assume eventually the period of the periodic force. If the impressed period approximately equals that of the locality, and is in phase, the phenomenon called *synchronism* occurs, and the locality vibrates through abnormally large amplitudes.

The factors which determine ground amplitudes during an earthquake are: the character of the invisible elastic earth waves, the dis-

FIG. 2—HORIZONTAL SLIP. FENCE CONTINUOUS PRIOR TO SLIP. FROM CALIFORNIA EARTHQUAKE INVESTIGATION COMMITTEE REPORT

FIG. 3—VERTICAL SLIP. ROAD CONTINUOUS PRIOR TO SLIP. FROM NEW ZEALAND JOURNAL OF SCIENCE

tance of the site from the fault, the nature of the strata, and the dominant period of the site.

The amplitude, period, and regularity of the invisible seismic waves affect the ground amplitude. The larger the wave amplitude, the closer the synchronism with the dominant period of the locality, the more regular the waves—the larger will be the ground amplitude.

The proximity of the site to the fault along which slip occurred also influences the ground amplitude—the closer to the fault, the larger the amplitude.

Again, other factors being equal, the ground amplitude will depend on the nature of the strata. Bedrock will vibrate least; dry, cohesive soil more; and wet, unconsolidated soil most. The amplitude of a wet soil will be larger than that of a similar but dry soil. The greater amplitude of a soft layer will not be prevented by an overlying thin stratum of drier and more compact material.

Finally, a site will respond much more readily, and with much greater amplitude when jarred by a synchronous earthquake, than by one otherwise similar but non-synchronous.

During an earthquake, the ground vibrates in all three dimensions. Its space orbit is very complex. As a matter of fact, seismographic records reveal that the ground motion is complex even if vibration in one plane is considered. In the majority of cases, the projection of the space orbit on an horizontal or vertical plane discloses crude circular or elliptical paths, irregular in form, and with abrupt changes in direction.

Excluding the case of synchronism, the maximum ground amplitude during a destructive earthquake is probably 0.25 inch for bedrock, 1.00 inch for dry cohesive soil, and 4.00 inches for wet unconsolidated ground.

Fig. 4 shows the horizontal vibration, during the Tokyo earthquake of September 1, 1923, of the ground occupied by the Seismological Institute. This earthquake, with the ensuing fire, caused 141,720 deaths, and the destruction of 580,397 buildings. The vibratory character of the motion, and the direction reversals are readily apparent. The site consisted of well consolidated fill. The average period of the ground vibrations during the most destructive interval, which occurred between the twelfth and seventeenth seconds from the beginning of the earthquake, was 1.3 seconds, and the amplitude was 1.9 inches.

It is on evidence such as this that an important part of the mathematics of dynamical aseismic design is based. For, although the vibrations shown in Fig. 4 are only crudely harmonic, it can be shown[9]

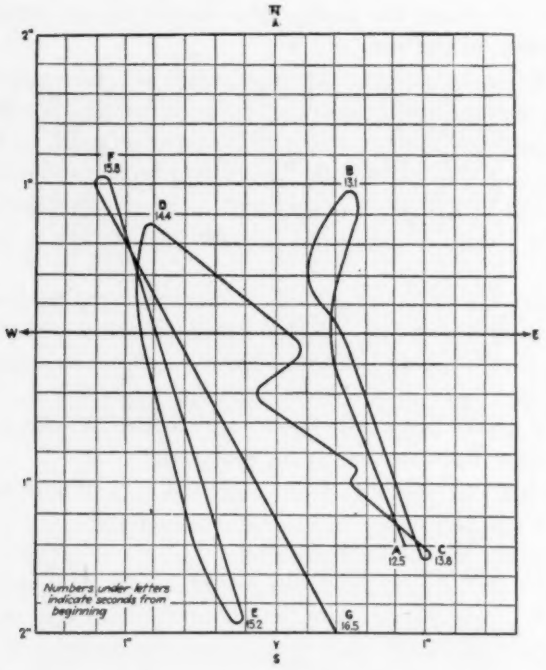


FIG. 4—HORIZONTAL VIBRATION OF SITE TOKYO EARTHQUAKE, SEPTEMBER 1923

that even when it is difficult to ascertain the precise character of a particular vibration, a close approximate solution can always be obtained by assuming a type which can be judged on independent grounds as being a fair representation of the unknown vibration.

Thus, with the ground acting as if it is approximating harmonic motion, there can be no great error involved in assuming that it is vibrating in harmonic fashion. Particularly so, when it is realized that this hypothesis fixes the most unfavorable limit for building resistance. After all, engineers and architects are accustomed to design structures to meet the most unfavorable combination of probable loading conditions.

STRUCTURAL VIBRATIONS

A structure also vibrates in dominant periods peculiar to itself. Its largest free period, for any particular mode of vibration, is called the *fundamental;* the next largest, the *first harmonic.* The largest amplitude is attained ordinarily while vibrating in the fundamental period.

Fig. 5 shows the vibration of the Washington Monument[10] caused by wind gusts of 20-30 mile per hour velocity. Its fundamental period is 1.81 seconds, the first harmonic 0.60 second, and the amplitude of vibration at the 500-foot level is approximately 0.0005 inch.

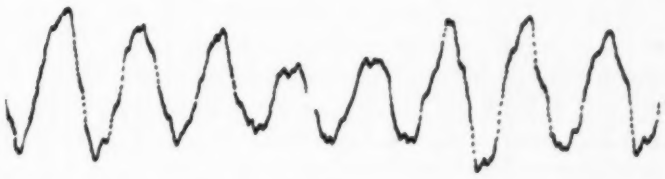


FIG. 5—FREE VIBRATION OF WASHINGTON MONUMENT RECORDED BY NEUMANN EXPERIMENTAL SEISMOGRAPH

When an earthquake occurs, a building is thrown into vibration. If its fundamental synchronizes with the impressed period, the building will vibrate through dangerously large amplitude. The seismic amplitude A of the building is a function of the following factors:

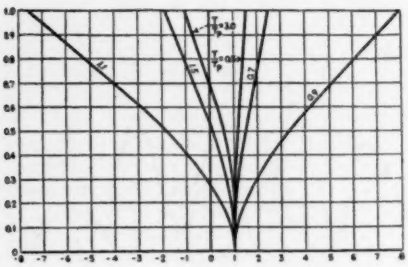
$$A = f\,(Q, C, T, S)$$

Q represents the period and amplitude impressed by the earthquake and site.

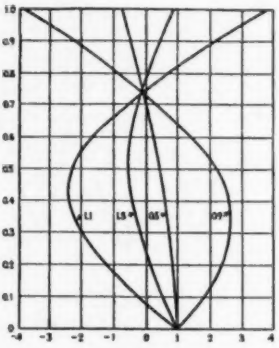
C, the end conditions factor, interprets the end fixity of the building. C depends on the compressibility of the ground and the restraint which it exerts on the building foundation. A building with its foundation *fixed* in bedrock will have a different value of C than a building whose foundation is in compressible soil, and therefore *hinged*. Fig. 6 shows the unit deflection, moment and shear curves for a tall building fixed at the foundation; Fig. 7 shows the corresponding curves for a building hinged at the foundation. Comparison of Fig. 6 and 7 illustrates how vitally C affects the character of the deflection, moment and shear curves.

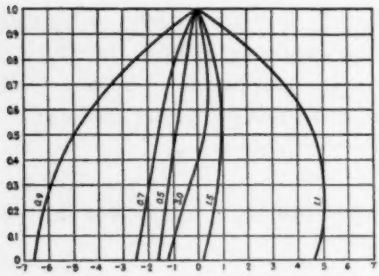
T, the fundamental period of vibration[11] is a function of C, of the physical dimensions of the building, its elastic resistance, the dead and live load distribution, and the acceleration due to gravity.

S, the synchronism factor, is the ratio between the fundamental period T and the impressed period T_p, or $S = T/T_p$. Fig. 6 and 7 illustrate the effect of S on seismic deflections, moments and shears. For example, Fig. 6 shows that when the building period is one-half

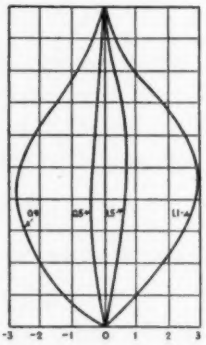


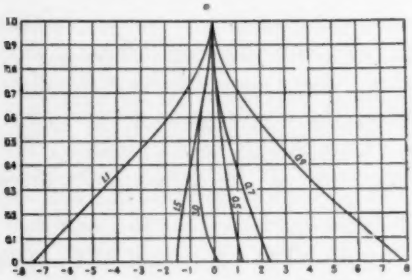
Deflection





Moment





Shear

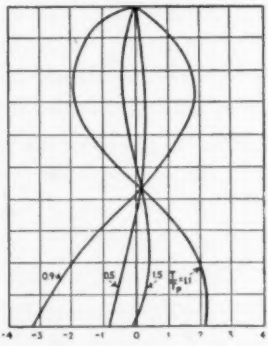


FIG 6—FOUNDATION FIXED FIG. 7—FOUNDATION HINGED

Unit deflection, moment and shear curves for various end conditions C, and various values of synchronism factor S, during forced vibration.

that of the seismic period, or $\frac{T}{T_p} = 0.5$, the deflection of the top of the building is 0.5. But, when synchronism is approached, or $\frac{T}{T_p} = 1.1$, the deflection is 8.5, or 17 times that in the former case. This demonstrates the profound influence of synchronism on building amplitudes.

EARTHQUAKE DAMAGE

A composite picture of earthquake damage reveals that, in modern buildings, the amount of damage parallels the degree of synchronism between the building and earthquake periods. Thus, in the Tokyo earthquake with a period of 1.3 seconds for the most destructive ground vibrations, buildings with periods less than 0.5 second fared better than those with larger periods.

Buildings of square or rectangular plan are, in general, damaged less than those of *E*, *H*, *L* or *U* plan. Frequently, failure occurs at the junctures of the wings to the main structure. Buildings with wall openings assymmetrical suffer greater damage than those with symmetrical arrangement.

One of the most frequent causes of collapse is an inadequate foundation. Differential settlement in the foundation nullifies the value of a good superstructure. Buildings not anchored to their foundations suffer damage.

Buildings so framed and braced that they vibrate as units with their foundations do well. Unbraced buildings collapse. Adjoining buildings hammer each other, but also tend to lend each other mutual support Unanchored roof trusses slide off the walls.

Poorly designed steel, or reinforced-concrete frames fail. Columns and beams rip apart. Column flanges tear loose. Knee braces shear off. Floors crack and settle. Facing, veneers, cornices, parapets, chimneys, and ornamentations crumble and fall. Walls and partitions crack, fail, or fall out of the frames. Rigid walls, weak in tensile strength, cannot endure the loads forced on them during vibration. This is illustrated in Fig. 8, which shows the X cracks suffered by the rigid walls of the steel-frame Japan Oil Building during the Tokyo earthquake.

Sixteen important steel-frame buildings were shaken by the Tokyo quake.[12] Of these, ten were more or less seriously damaged; one practically destroyed. The remaining six were damaged either slightly, or not at all. It is a noteworthy fact that the ten seriously damaged buildings used materials of negligible tensile and shearing strength for their walls, whereas the six undamaged buildings used reinforced-concrete walls extensively.

FIG. 8 (UPPER LEFT)—X-CRACKS IN RIGID BUT WEAK WALLS. STEEL FRAME JAPAN OIL BUILDING. TOKYO EARTHQUAKE, SEPTEMBER 1923. AFTER J. R. FREEMAN

FIG. 8A (UPPER RIGHT)—REINFORCED-CONCRETE COLUMN FAILURE. LONG BEACH, CALIFORNIA EARTHQUAKE, MARCH 1933. COURTESY NATIONAL BOARD OF FIRE UNDERWRITERS

FIG. 8B (LOWER LEFT)—REINFORCED-CONCRETE WAREHOUSE. CRACKS IN RIGID BUT WEAK CONCRETE WALLS. LONG BEACH, CALIFORNIA EARTHQUAKE, MARCH 1933. COURTESY NATIONAL BOARD OF FIRE UNDERWRITERS

FIG. 8C (LOWER RIGHT)—JEFFERSON JUNIOR HIGH SCHOOL, LONG BEACH, CALIFORNIA EARTHQUAKE, MARCH 1933. COURTESY NATIONAL BOARD OF FIRE UNDERWRITERS

Measuring the periods of vibration[13] of buildings both before and after earthquakes, and comparing the values for the two cases, indicates that in seriously damaged buildings periods increase; in undamaged structures they remain substantially unaltered.

Analysis of earthquake damage indicates that only structural materials which can withstand the bending reversals of vibration are practicable in earthquake-resistant construction. However, regardless of the materials used, they must be of good quality, the design must be specific and adequate, and the construction honest and competent.

ASEISMIC DESIGN

In the foregoing discussion, the observational, experimental, and theoretical considerations affecting the earthquake problem have been outlined. The solution is implicit in the data, and consists of applying rationally the deductions obtained therefrom.

In earthquake-resistant design, seismological and geological investigations are required. The proximity of faults must be determined. It would be very unwise to locate a building over a known fault; for,

FIG. 8D—HELENA HIGH SCHOOL—$500,000 STRUCTURE. COMPLETED JUST IN TIME FOR THE MONTANA EARTHQUAKES, OCTOBER 1935

FIG. 8E—HELENA HIGH SCHOOL. AFTER THE EARTHQUAKES OF OCTOBER 1935. COURTESY U. S. COAST AND GEODETIC SURVEY

in the event of slip it is probable that the building will be destroyed. Fig. 9 illustrates the fate of a building which straddled a fault.

The dominant period of the site must be measured, in order that the proposed building will be designed with periods non-synchronous with that of the locality, or permit the use of the correct synchronism factor in the site amplitude. The dominant period of the site may be obtained by measuring with seismographs the vibrations set up by traffic, by dynamite blasts, or by a shaking machine operating through a varying R. P. M. All three methods[14] should be used and the results checked against one another. In the case of the shaking machine, increasing amplitude on the record indicates the approach of synchronism with a dominant period of the site. Fig. 10 shows the synchronism curves obtained during forced vibration of the San Francisco Mint site. Exact synchronism is not shown, the dominant period of the site being outside the effective R.P.M. range of the shaking machine. However, the fact that the amplitude increases abnormally when the site is shaken at a rate of 600 R.P.M., or in a period of 0.10 second, indicates that the dominant period of the site is somewhat smaller than 0.10 second.

Inasmuch as the nature of the materials constituting a site determines its seismic amplitude, and affects the end conditions of the building, the necessity of reliable geological data is apparent.

A correctly designed foundation is a vital factor in the ability of a structure to survive a major earthquake undamaged. The ideal aseismic foundation is a beam and slab mat. This type not only cancels the adverse effect of differential soil movement, if the site is not of uniform composition, but also lends itself to a rigid and monolithic design. If individual footings must be used, these should be inter-connected by beams able to maintain them in the same relation position during vibration. Where the building is supported on piling, the piles should be tied together by a beam and slab mat. Short piles, which permit the building to rotate in a vertical plane, are favored where practicable.

Satisfactory aseismic design requires symmetry of mass and rigidity. The center of mass should coincide with the center of rigidity, otherwise, adequate provision should be made for the resulting torisonal forces about the center of mass. Buildings should preferably be of closed plan, and separated from adjoining structures. Rigid bents should be symmetrical about the floor plan. All parts of the structure and its foundation must be so connected to the frame that the building will vibrate as a unit, with connections strong enough to

FIG. 9—BUILDING ON FAULT—AFTER SLIP. FROM CALIFORNIA EARTHQUAKE INVESTIGATION COMMITTEE REPORT

overcome the inertia of the parts. Wings should be designed for rotational forces, or else freed.

Floors act as horizontal girders during vibration and should be designed as such. The floor system should be so braced that it will deflect as a unit, and will be strong enough and rigid enough to transmit the lateral forces to all the bents. Floor slabs must be of reinforced-concrete, rigid and monolithic.

Rigid exterior and interior walls must be of adequate strength, located symmetrically on the plan, and designed as vertical beams. Wall bents should be continuous to the bottom of the building. Metal lath partitions, fastened to the frame, are favored. Chimneys must be of reinforced-concrete, or of sheet metal. Construction joints should be doweled, as this increases their reliability.

If rigid but weak materials are not to fail, either the seismic deflections of the structural members should not exceed the allowable limits

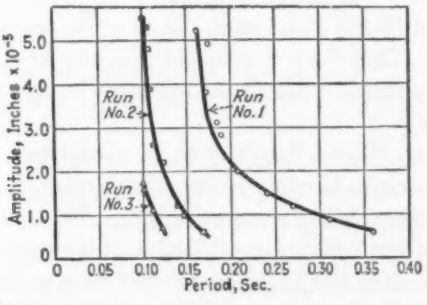


FIG. 10—FORCED VIBRATION OF SAN FRANCISCO MINT SITE

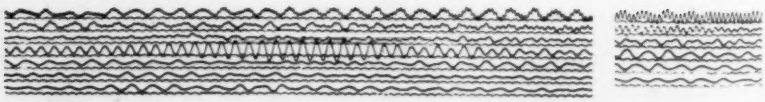


FIG. 11—FORCED VIBRATION OF BANK OF AMERICA BUILDING. COURTESY U. S. COAST AND GEODETIC SURVEY

for walls, partitions, veneers, cornices, parapets or ornamentations, or else these items should be freed from the frame. Trusses should be knee-braced, and built into the supporting columns. Close attention to details is a fundamental rule in aseismic design.

Building periods are required in order to evaluate the synchronism factor between ground and building vibrations. A building having been designed for its static loading, and in accordance with the foregoing principles, the periods of vibration are computed from the plans.[11] The effects of earthquakes of various periods, and of amplitude consistent with the site characteristics and periods are calculated, and the structural elements of the building are analyzed for adequacy against the maximum seismic deflections, moments, and shears.

The periods of existing buildings can, of course, be obtained by measuring the vibrations set up by wind, by traffic, or by shaking machine. Fig. 11 shows the forced vibration, parallel to Santa Clara Street, of the Bank of America Building, San Jose, California, under the action of a shaking[15] machine. In this case again, the abnormal amplitudes of synchronism give the dominant periods of the building. These are: 1.33 second, fundamental; 0.38 second, first harmonic.

It has been said that comparing the periods of a building taken before and after an earthquake provides a method of checking on seismic damage. It may be recalled that the period of vibration is a function of the end conditions, the dimensions of the building, its elastic resistance, its loading, and the acceleration due to gravity. Considering these five factors, it is evident that the dimensions of the building, its loading, and gravity are constants. It is improbable[16] that the end conditions vary. Therefore, if the periods are to increase, the elastic resistance of the building must decrease. This holds even in the absence of visible damage.

It is of interest to note that the U. S. Coast and Geodetic Survey has measured the periods of more than 200 important buildings[17] in California. It is intended to remeasure these periods after the buildings have been shaken by a major earthquake. Comparison of the periods taken before and after the earthquake will then provide a mechanical check on the structural conditions of the buildings.

The Coast and Geodetic Survey is much to be commended for its earthquake, site, and building vibration investigations. The accumulated data are of great importance to engineering seismology and the work should be encouraged.

DYNAMICAL VS. STATICAL DESIGN

The seismic deflections, moments, and shears may be estimated by either the dynamical or statical method. The statical method assumes that a building is a vertical cantilever subjected, during an earthquake, to a static horizontal force equal to a certain percentage of the dead and live loads above the section under consideration. The percentage chosen depends on the proximity of active faults, and on the character of the soil. The only other consideration involves the choice of "rigid" or "flexible" construction.

In common with the statical method, the dynamical procedure investigates proximity to faults, soil characteristics, and building loading. In addition, however, it considers the effects of site periods, end conditions and synchronism between the building and earthquake periods. All of these factors are of first order importance in determining the locations and amounts of the maximum seismic deflections, moments, and shears.

The question arises: which method, dynamical or statical, more nearly represents reality. Comparison of earthquake damage with theoretical expectations furnishes the answer. For example, Freeman[18] reports that for equal earthquake motion, a carefully designed rigid building on soft ground suffers much less damage than a similar building on rock. Martel[19] points out that buildings on soft ground were damaged less than those on good soil. Naito[20] observes that the most vulnerable point for building damage is at 0.30 the height, but that there are may exceptions.

By statical standards all of these observations are paradoxical. For, if buildings were vertical cantilevers, the maximum seismic moments and shears should occur theoretically at the bottom. It follows that the maximum seismic damage should also center at this point. Also, from the statical viewpoint, for equal earthquake motion, similar buildings should be damaged equally whether on soft ground or on rock.

Under the dynamical method, however, these so-called paradoxes are no more than what could be expected theoretically. In general, for equal ground motion, similar, well-designed buildings should suffer less damage on soft than on rigid ground. Fig. 6 and 7 illustrate this point, as well as the fact that the maximum moment may occur at any point 0.00 and 0.43 of the height, and the maximum shear between

0.00 and 0.75 of the height. Hence, the maximum earthquake damage may center at any plane of weakness between the bottom of the building and 0.75 its height.

It now seems to be generally accepted that the aseismic design of a tall structure is a dynamical problem. However, some engineers and architects, attracted by the relative simplicity of the statical theory and believing it safe, maintain that a building of moderate height can be considered as a "simple" rigid structure, and may therefore be designed statically.

There is, however, no fundamental criterion, other than size, by which a low earthquake-resistant building differs from a tall one. It is true, of course, that the deflection curve of a low building is accounted for mainly by deflections due to shear, whereas that of a tall building is due principally to moment. In both cases, however, the deflection curves result from some combinations of shear and moment. Furthermore, the deflection curve of the low building as well as of the tall one is affected by end conditions; both have periods of vibration; both respond to synchronism.

Simplicity in design is much to be desired, but not if it is attained by ignoring fundamental principles. It is valid only if all of the primary influences are represented. The fatal weakness of the statical method is that, although buildings designed statically are of necessity made arbitrarily strong and rigid to take care of the unrepresented factors, so that safe designs are sometimes accidentally obtained, the final result is always in doubt. There is no way for the engineer or architect to know when the design has ventured into dangerous territory. On the other hand, the dynamical method, even in its present state of flux, gives a correct qualitative picture. It produces designs which are safe, and only as strong and as rigid as the case data require. It would be well, in the interest of public safety, to check statical designs by dynamical analysis.

RIGID VS. FLEXIBLE DESIGN

The theory of rigid design, accepted by a majority of the statical school, is based primarily on Japanese experience that rigid, low-period buildings suffered less damage than structures of larger periods. It was reasoned that the better performance of the rigid buildings was due to their negligible synchronism with destructive earthquakes, which were believed to occur only in a bracket of periods ranging from 1.0 to 1.5 seconds.

If such a bracket really exists, then rigidity might be the answer to aseismic design in Japan, where building heights are limited to 100 feet. It would hardly do for buildings much taller than 100 feet, their

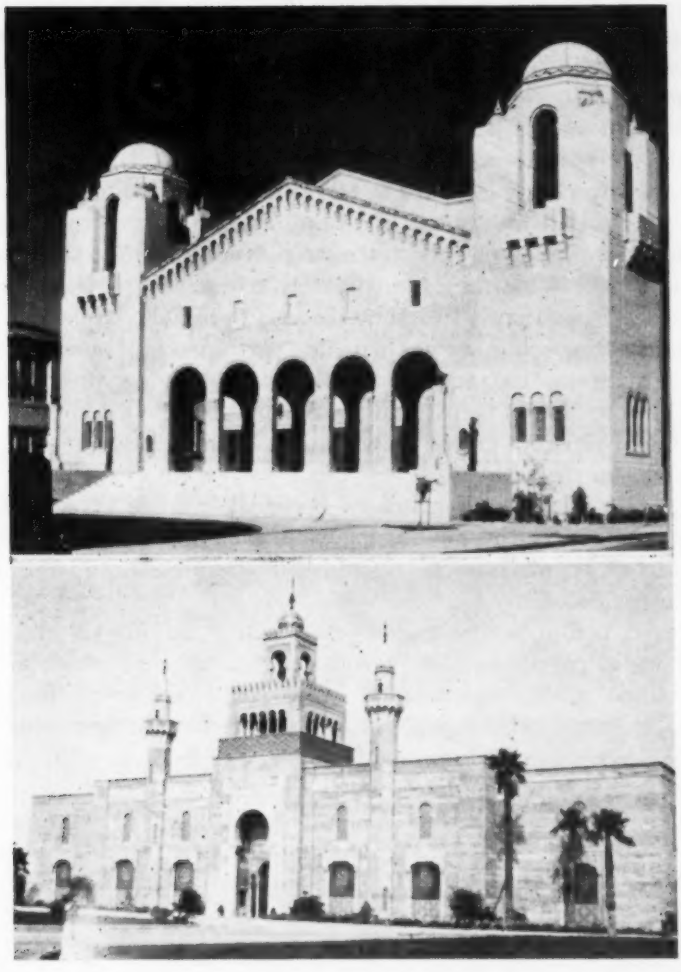


FIG. 12—AUDITORIUM, POLYTECHNIC HIGH SCHOOL. AFTER LONG BEACH EARTHQUAKE OF MARCH 1933. REINFORCED-CONCRETE CONSTRUCTION WITH STEEL ROOF TRUSSES. SLIGHTLY DAMAGED. COURTESY NATIONAL BOARD OF FIRE UNDERWRITERS

FIG. 13—MAUSOLEUM. AFTER LONG BEACH EARTHQUAKE OF MARCH 1933. REINFORCED-CONCRETE CONSTRUCTION. UNDAMAGED. COURTESY NATIONAL BOARD OF FIRE UNDERWRITERS

periods being either among, or greater than those in the dangerous bracket of periods.

However, the reality of any such bracket is highly questionable. Destructive earthquakes, of course, have had periods in this bracket; for example, the Tokyo earthquake of 1.3 second period. But major earthquakes of other periods have occurred; for example, Long Beach 1933, of 0.30 second period; and Helena 1935, of 0.13 second period. In fact, there is cause to believe[21] that destructive earthquakes may have any period included in the entire range of building periods.

The reason that low-period, rigid buildings fared well in the Tokyo earthquake was due not only to their rigidity, but also to the fact that synchronism between the approximately 0.3 second period buildings, and the 1.3 second period earthquake was negligible. Whether these same structures would survive a major earthquake of short period is another question, the answer to which depends on their inherent strength, as well as on their rigidity. Strange as it may seem, excessive and random rigidity, unless accompanied by requisite strength, may do more harm than good.

A minority of the statical school favor the flexible type of design. This is based on the theory that a flexible zone in a building, generally a high first story, will ease and decrease the seismic motion transmitted to the upper floors.

However, before flexible construction can function, the rigid contiguous walls, partitions and trim in the flexible zone must fail. Of course, these rigid elements may be designed free from the frame. Even so, it seems quite a problem to design a flexible zone which can take the tremendous seismic loads and still remain flexible. Structural collapse due to foundation movement is a possibility. It should also be kept in mind that the flexible method does not make positive provision for preventing damage to the upper part of the building.

Buildings deficient in strength and rigidity are dangerous; those with strength and rigidity above requirements are at best uneconomical. Structures of strength sufficient to take the induced seismic loads, and of rigidity adequate to protect their contents, are the goal in modern aseismic design.

COST OF ASEISMIC CONSTRUCTION

At first thought it would seem that earthquake-resistant construction must be expensive. Actually, however, the cost is little more than that of ordinary modern construction. Experience shows that statically designed buildings rarely exceed by five per cent the cost of structures with no aseismic features. For buildings of dynamical design, the cost may be somewhat greater because of the geological,

seismological and analytical investigations required. In any case, the cost depends on the magnitude of the project, and the conditions to be met.

Sometimes all that is necessary is a rational evaluation of the seismic forces, and the intelligent use and combination of the available structural elements and materials. In one case, a $2,000,000 project, the cost of the dynamically designed building did not exceed the estimate for the ordinary design.

In any event, a cost increase of a few per cent is certainly not a prohibitive price to pay for assuring safety to tenants and capital investment. Particularly so when it is considered that the additional cost is gradually refunded in the form of savings due to lower earthquake insurance rates.

REINFORCED-CONCRETE IN ASEISMIC DESIGN

The characteristics of reinforced-concrete which make it a particularly happy choice for use in earthquake-resistant construction will now be analyzed.

Some of these are obvious, and will not be discussed. For example, reinforced-concrete has behaved well in past earthquakes, (Figs. 12 and 13) is well adapted to monolithic construction, and to producing buildings which vibrate as units. Not so apparent are other characteristics, such as its value in damping vibrations; its aseismic potency when used in floors, walls and partitions; and its efficiency in limiting vibrations to amplitudes which will not injure rigid but weak materials.

REINFORCED-CONCRETE IN DAMPING VIBRATIONS

Of the two principal structural materials, reinforced-concrete and structural steel, the one with the greater damping value will resist vibration better, will have less amplitude during synchronism, and will reduce its amplitude to negligible proportions sooner.

To determine qualitatively the relative damping values of reinforced concrete and structural steel, theoretical, rectangular cantilever beams of the two materials will be compared. To simplify calculations, the beams are assumed to be designed to carry equal uniform loading, and to be identical in width and span, but of different depths.

The fundamental period of vibration of a cantilever beam may be expressed as

$$T^2 = 3.20 \frac{wL^4}{EIg} \qquad \text{.....(1)}$$

where T = fundamental period, seconds
W = weight, lbs. per inch of L
L = length, inches
E = Young's modulus for the material, p.s.i.

I = moment of inertia of the section perpendicular to the plane of vibration, in.^{4}
g = acceleration due to gravity, 386 in. per sec.^{2}

Hence, if the subscripts $_c$ and $_s$ denote concrete and steel respectively,

$$\frac{T_c^2}{T_s^2} = \frac{w_c E_s I_s}{w_s E_c I_c} \quad \ldots\ldots (2)$$

Let M = bending moment at any point x, in.-lbs.
s = extreme fiber stress in compression, lbs. per in.^{2}
b = width of beam, in $b_c = b_s$
c = one-half the depth d, in.

then $M = \frac{wx^2}{2}$ $s = \frac{Mc}{I}$ $I = \frac{bd^3}{12} = \frac{2bc^3}{3}$

and $$c^3 = \left(\frac{3wx^2}{4bs}\right)^{\frac{3}{2}} \quad \ldots\ldots (3)$$

By substitution, eq. (2) becomes,

$$\frac{T_c^2}{T_s^2} = \frac{E_s}{E_c}\left(\frac{w_s}{w_c}\right)^{\frac{1}{2}}\left(\frac{s_c}{s_s}\right)^{\frac{3}{2}} \quad \ldots\ldots (4)$$

The damping factor [7] for a cantilever beam is given as

$$e^{-\frac{1}{2}kt} \quad \ldots\ldots (5)$$

where $$k = \frac{4\pi^2 R}{T^2 E} \quad \ldots\ldots (6)$$

and e = Napierian base = 2.718+
R = coefficient of damping
t = time from beginning of vibrations, sec.

Therefore $$\frac{T_c^2}{T_s^2} = \frac{E_s k_s R_c}{E_c k_c R_s} \quad \ldots\ldots (7)$$

Equating (4) to (7),

$$\frac{k_c}{k_s} = \frac{R_c}{R_s}\left(\frac{w_c}{w_s}\right)^{\frac{1}{3}}\left(\frac{s_s}{s_c}\right)^{\frac{3}{2}} \quad \ldots\ldots (8)$$

Experimental values [22] of $R_c = 7270$, $R_s = 164650$. It will be assumed that $s_c = 700$, and $s_s = 18000$. The beams were designed to carry identical loading. Hence $d_c = 5.07 d_s$. Assuming $b_c = b_s = 12$ in., and $d_s = 12$ in., then $d_c = 60.84$ in. From (3) the values of w for vibration as fully loaded beams are: $w_c = w_s = 180$ p.s.i.

Substituting for R, w, and s in eq. (8), it is found that

$$k_c = 5.76\, k_s \quad \ldots\ldots (9)$$

I_s, w_s, R_s, and g are given. Assuming, for purpose of illustration, L = 240 in., $t = 1$ sec., and calculating $T_s^2 E_s$ by eq. (1), then by eq. (6), $k_s = 2.26$. Therefore, by (5) the damping factor of the structural steel beam is

$$e^{-\frac{1}{2}k_s t} = e^{-1.13} \quad \ldots\ldots (10)$$

and that of the reinforced-concrete beam is

$$e^{-\frac{1}{2}k_c t} = e^{-6.51} \quad \ldots\ldots (11)$$

Obviously the damping factor of the reinforced-concrete beam is the larger. This means that the reinforced-concrete beam is more effective in resisting vibration, vibrates with less amplitude during synchronism, and comes to rest sooner than the structural steel beam.

REINFORCED-CONCRETE IN FLOORS AND WALLS

In any of the conventional methods of building bent analysis, the frame only is investigated. A building, however, resembles its frame only remotely. There is no simple relationship, and little similarity between the deflection of a building and that of its frame, for the reason that the deflections of the horizontal structural members are limited by the floor slabs, and those of the vertical members are modified by the walls.

How important the restraining influences of the slabs and walls are in determining the deflection behavior of a building can be realized from the following considerations: In an unloaded frame, with known end conditions, the computed deflection under a specified lateral load will be found to check the measured deflection. But, if the dead and live loads representing conditions of usage are added to the frame, the new computed deflection may be several hundred percent greater than the deflection of the building. Clearly the slabs and walls are acting other than as just so much dead load.

Again, if building periods are computed without taking into account the stiffening effect of the slabs and walls, they will be found to be much larger than the measured periods of vibration.

In both cases, if the stiffening influence of the slabs and walls is considered, the computed deflections and periods will check reasonably well with the measured deflections and periods.

Obviously, the slabs and walls influence the rigidity and strength of a building. Reports on earthquake damage suffered by buildings which did not collapse, uniformly agree that slabs and frames were damaged slightly, but that walls suffered considerable damage. The preponderance of data is to the effect that slabs acted as stiff horizontal girders, that walls were in full action while columns were not, and that until the walls failed, they and not the columns bore the brunt of the lateral thrust.

These facts are not new. Certainly they are in agreement with common sense. It is evident that a frame cannot begin to function as designed until the contiguous slabs and walls have failed. Nevertheless, in spite of the very considerable experience which shows that the assumption does violence to the physical facts, current practice continues to assume that slabs and walls play no part in resisting

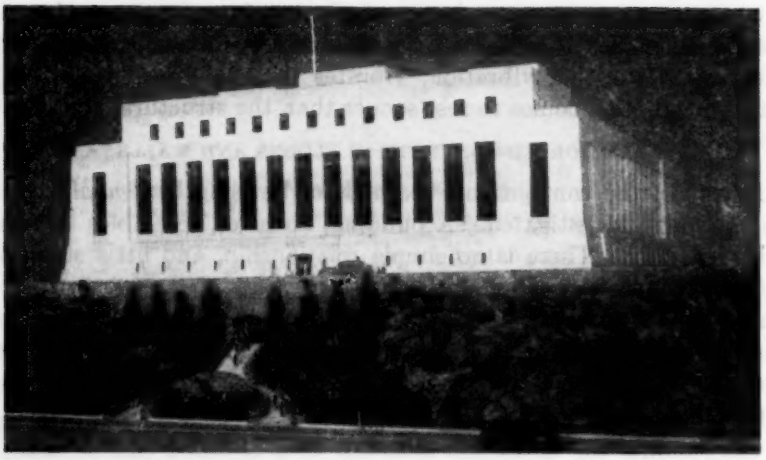


FIG. 14—EARTHQUAKE-RESISTANT SAN FRANCISCO MINT REINFORCED-CONCRETE AND STRUCTURAL STEEL CONSTRUCTION. COURTESY PROCUREMENT DIVISION, TREASURY DEPARTMENT—G. S. UNDERWOOD, CONSULTING ARCHITECT; JACOB J. CRESKOFF, CONSULTING ENGINEER

earthquake forces. It would seem the part of good judgment to design slabs to distribute the forces falling to them as horizontal girders; and to design walls to withstand uninjured the loads forced on them by reason of their rigidity.

Reinforced-concrete is unexcelled for these purposes. Its use in floors produces horizontal girders which deflect as units. Its use in walls makes it possible to design these strong enough to take seismic loading safely. With the help of reinforced-concrete, buildings are better able to endure major earthquakes undamaged, rather than surviving merely as frames carrying the wreckage of masonry.

REINFORCED-CONCRETE IN CONTROLLING AMPLITUDE

During an earthquake, the seismic force acts against the mass of a building. Assuming that the building mass is concentrated at the floor levels—which errs on the side of safety—and considering the floors as horizontal girders which deflect as units, then the columns and walls between two consecutive floors may be regarded as vertical beams, deflecting equally because of a concentrated horizontal force acting against the upper floor, and with end conditions determined by the degree of fixity between the vertical beams and the floors. Under these assumptions[23] the seismic force is distributed to all of the vertical beams, and to each in proportion to its relative rigidity.

A typical analysis by the method of relative rigidity reveals why walls are forced to do most of the work in resisting earthquakes.

For example, in the case of the San Francisco Mint, (Fig. 14), the seismic force is divided among the various structural elements as follows:

Between the first and second floors, 129 structural steel columns take 0.3 per cent, and 106 reinforced-concrete vertical beams (walls and partitions) 99.7 per cent of the seismic force.

Between the second and third floors, 8 K-braced steel bents take 31.6 per cent, 113 steel columns 5.6 per cent, and the reinforced-concrete vertical beams 62.8 per cent.

Between the third and fourth floors, 8 K-braced steel bents take 12.7 per cent, 100 steel columns 1.8 per cent, and 90 reinforced-concrete vertical beams 85.5 per cent.

Evidently the steel columns, although efficient against vertical loads, are quite inefficient against horizontal loading. During earthquakes, the braced bents and the reinforced-concrete walls and partitions will do most of the work.

Consideration of the method of relative rigidity reveals that if the rigidities of the more rigid elements are decreased, the less rigid elements become more efficient, and vice versa. The apportionment of rigidity among the structural members depends, of course, on the amount of deflection permitted.

The use of reinforced-concrete provides an efficient method of minimizing deflection, because not only walls but also partitions may be used. This type of bracing does just as well whether applied to structural steel or to reinforced-concrete frames.

CONCLUSIONS

There is probably no part of the United States which has not experienced destructive earthquakes, although some sections are at present more active than others. No region, however, can be assumed to be immune from the possibility of earthquake damage.

The average seismic frequency during the past three centuries has been as follows: disastrous earthquakes, 1 in 10 years; earthquakes inflicting severe admage, 7 in 11 years; and earthquakes causing some degree of damage, 18 in 11 years.

Seismic activity occurs in roughly regular cycles. As active phase of the United States cycle is now being approached. It is probable that several important earthquakes will be experienced between the years 1939-1946.

Theory and observation dictate that the following principles be applied in the design of earthquake-resistant structures.

A building must not straddle a fault. It should have an integrated foundation, symmetry of mass and rigidity, and preferably a closed plan. It should be designed to vibrate as a unit, be sufficiently rigid to protect its contents, and strong enough to endure the induced seismic loads safely. Attention to details is a fundamental requirement.

Since the vibration of the building will depend on the intensity of the earthquake, the nature of the site, the end conditions of the building, its physical characteristics and loading, and on the degree of synchronism between the earthquake, site and building; the resultant of these effects must be evaluated.

Only materials of requisite tensile and shearing, as well as of compressive strength, are practicable in aseismic design. In addition, the materials must be of good quality, the design specific, and the construction competent.

Designed along these principles a building, although costing a bit more than ordinary construction, has little to fear from earthquakes.

Measuring and comparing the periods of vibration of a building before and after an earthquake affords a mechanical check on structural changes. This method is valuable for detecting invisible earthquake damage.

Reinforced-concrete is particularly suitable for use in the design of earthquake-resistant structures. It resists vibration, permits monolithic construction, and is unexcelled for foundations, floors, and walls. Correctly designed, it produces buildings which are rigid enough to protect the non-structural elements, and strong enough to survive violent earthquakes undamaged.

ACKNOWLEDGMENTS

The author is indebted to Huber B. Churchill, Secretary, Great American Insurance Company, Robins Fleming, Structural Engineer, American Bridge Company, Captain N. H. Heck, Chief, Seismology Section, U. S. Coast and Geodetic Survey, Leon Moisseiff, Consulting Engineer, and Commander Ben Moreell, Bureau of Yards and Docks, U. S. Navy, for constructive criticisms and suggestions.

REFERENCES

(1) N. H. Heck, "Earthquakes," Princeton University Press, 1936, p. 114.
(2) C. Davison, *Nature*, April 11, 1936, p. 605.
(3) N. H. Heck, "Special Bulletin No. 149," United States Coast and Geodetic Survey.
(4) N. H. Heck, "Special Bulletin No. 191," United States Coast and Geodetic Survey.
(5) J. R. Freeman, "Earthquake Damage and Earthquake Insurance," McGraw-Hill Book Co., 1932, p. 111.

(6)C. Davison, *Nature*, October 22, 1927, p. 587.
(7)K. Suyehiro, *Proceedings A.S.C.E.*, May, 1932.
(8)B. Gutenberg, "Special Bulletin No. 201," United States Coast and Geodetic Survey.
(9)J. W. S. Rayleigh, "Theory of Sound," Vol. I, p. 255.
(10)J. J. Creskoff, "The Washington Monument," *Journal of the Franklin Institute*, November 1934, p. 533.
(11)J. J. Creskoff, "Dynamics of Earthquake Resistant Structures," McGraw-Hill Book Co., 1934, Chapter 9.
(12)H. M. Hadley, "Bulletin, Seismological Society of America," March, 1924, p. 6.
(13)J. J. Creskoff, "Periods of Vibration: An Index to Structural Integrity," *Engineering* (London), June 14, 1935, p. 618.
(14)J. J. Creskoff, "San Francisco Mint Designed to Resist Earthquakes," *Engineering News-Record*, Oct. 3, 1935, p. 466.
(15)J. A. Blume, "Special Bulletin No. 201," United States and Coast Geodetic Survey.
(16)J. J. Creskoff, "Periods of Vibration: An Index to End Conditions," *Journal of the Franklin Institute*, May, 1935, p. 573.
(17)D. S. Carder, "Special Bulletin No. 201," United States Coast and Geodetic Survey.
(18)J. R. Freeman, "Earthquake Damage and Earthquake Insurance," p. 767.
(19)R. R. Martel, "Special Bulletin No. 201," United States Coast and Geodetic Survey.
(20)T. Naito, "Earthquake Resisting Construction," (In Japanese. Translation available in A. S. C. E. library) p. 279.
(21)A. Blake, "The Recording of Strong Seismic Motion," Bulletin, Seismological Society of America, July, 1933, p. 111.
(22)K. Suyehiro, "Bulletin Imperial Earthquake Research Institute of Japan," Vol. VI, p. 63.
(23)J. J. Creskoff, "Earthquake and Wind Design: Suggested Rationalization," *Engineering News-Record*, November 1, 1934, p. 553.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by April 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for May-June, 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY 1937

Maintenance and Repair of Concrete Structures in Railroad Construction*

By M. Hirschthal†
MEMBER AMERICAN CONCRETE INSTITUTE

As recently as 25 years or even 20 years ago, whenever an important permanent structure or a series of structures in an improvement was projected by the Delaware, Lackawanna & Western Railroad, comparative estimates would be made for a structural steel and a corresponding reinforced concrete structure. In making these estimates there would be added to the estimate of steel construction (or credited to the cost of the reinforced concrete structure) the item of the capitalized cost of maintenance of the steel structure; in other words, the cost of painting the steel structure. (For instance: if it were decided that a particular structure required painting every three years and money was worth five per cent, the cost of painting divided by three and multiplied by twenty would be added to the estimated cost of the steel construction for comparison with the concrete structure.) This obviously means that the basis of the comparison implied that it would be unnecessary to spend any money on maintenance of the concrete structure. On the other hand, there was waived the item of the cost of reinforcing the steel structure in the future for increased locomotive and other loadings, whereas it was correctly assumed that the concrete or reinforced concrete would have the advantage of gaining increased compressive strength with increase in age and due to the reduction of the ratio of the modulii of elasticity there would also be provided increased resistance on the tensile side of the reinforced concrete member. These were the considerations that led to the selection of concrete as the material for practically all of the structures (56 major structures) on the cut-off line from Port Morris,

*Received by the Institute Dec. 17, 1936 for presentation 33rd Annual Convention.
†Concrete Engineer, Delaware, Lackawanna & Western Railroad.

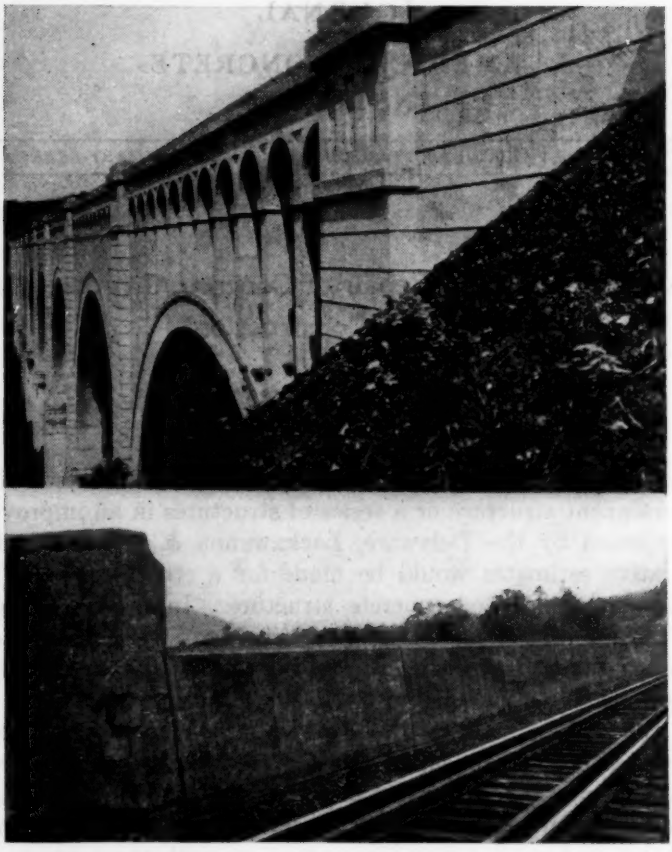


FIG. 1 AND 2—TUNKHANNOCK VIADUCT—THE "EIGHTH WONDER"—BUILT BY D. L. & W. RY., 1914; TEN 180 FOOT SPANS, TWO 100-FOOT SPANS, HEIGHT 240 FEET. THESE PHOTOGRAPHS MADE SEPTEMBER 1936

N. J., to Slateford, Pa. (known as the Lackawanna Railroad of New Jersey), 1908-1911, as well as for the cut-off line between Scranton, Pa., and Binghamton, N. Y. (84 major structures), built 1912 to 1915. This was also true in the selection for the construction of the Buffalo Terminal Improvements, 1915-1916, as well as for the grade eliminations at the east end of the line.

Typical of this are the Tunkhannock Viaduct at Nicholson, Pa. (on the Pennsylvania cut-off line), an arched viaduct almost one-half mile long with a height of 240 ft. from rail to creek; and the Buffalo Trainshed support, a flat slab viaduct 1,100 ft. long. Of course, the

superior architectural treatment possible was a consideration that certainly favored the concrete in this comparison.

Part of the scope of this paper is to report to what extent this assumption was justified,—the present condition and the amount of maintenance, if any, on the structures referred to, and others of similar character, after a lapse of 20 to 30 years.

The subject assigned to the author is "maintenance and repair of concrete structures." The question immediately presents itself when discussing concrete: "Where does maintenance stop and repair begin?" Maintenance, in the opinion of the author, constitutes such treatment immediately after the completion of the structure (or before any damage has occurred) which will maintain the integrity of the structure to carry the loads for which it is designed, as well as its architectural appearance, and the renewal of such treatment when the original application shows signs of deterioration or failure. This, of course, is the province of surface treatments. Evidently our assumption when designing structures was that such maintenance would be unnecessary.

A generation ago it was thought unnecessary to apply any surface treatment to a new structure. We on the Lackawanna Railroad always felt that the best and best-looking concrete was that which showed the form marks (aside from fins); surface washes were and still are frowned upon. It was only when a repair became necessary that any steps were taken for the correction of an existing concrete structure.

In all these years, aside from the granite protection of river piers, there has been but one attempt at maintenance within the definition stated above on any Lackawanna structure, and in this instance it resulted from the fact that its location was such that there was a probability of its being subjected to an acid-laden atmospheric condition. The structure in this instance is the Hackensack River concrete flat-slab approaches and even here it was only the columns (almost 50 ft. high) which were protected. The treatment consisted of an application of emulsified asphalt. It had been expected to cover the asphalt with a spray of cement immediately, the cement to unite with the water of emulsification in the course of setting, so as to eliminate the unsightly black appearance. Because of the difficulty of the cement application on these high columns, the cement spray was omitted.

Inability of concrete structures to maintain their initial integrity both as to strength and appearance is traceable either to design or installation. Under design are included of course, in addition to

provisions for stresses due to loads and soil bearing, provisions for drainage and the selection of location of expansion and construction joints so as to compensate for stresses due to volume and temperature changes—a most prolific source of crack formation. Included under installation are the design of concrete mixtures, the selection of the various materials including water, the workmanship of mixing and placing, and perhaps most important of all the curing of the concrete after it is placed. An error in any one of the above items is likely to result in the deterioration and disintegration of the concrete structures. It is only in recent years that it has been realized that strength and durability of concrete do not necessarily go hand-in-hand, although it has been many years since this was recognized in the case of irons and steels. Cements that meet strength requirements may not react favorably to the action of freezing and thawing in cycles. A series of tests now being conducted by the Highway Research Board on specimens made with the same aggregate but various types of cements already indicates a wide variation in resistance to cycles of freezing and thawing. These tests, while not conclusive since they are being made on specimens only 2 in. thick and not necessarily indicative of action on actual structures, still furnish valuable information as a guide in further research as well as in immediately calling attention to such variation. Considerably more research will be required to furnish results from which to draw definite conclusions on this subject.

Sands are ingredients on which there is available more definite information from investigations of their source, yet from the standpoint of durability our most recent specifications have developed a weakness insofar as they permit the use of sands with known undesirable constituents provided they meet a certain strength requirement. It is emphatically unwise to permit the use of such sands where durability is a requirement.

DURABILITY OF CONCRETE

We now come to the coarse aggregate as a factor in determining the durability of the concrete. With the practice of spading such that the aggregate is kept away from the face exposed to the weather, it would seem that it is only necessary for the mortar face protecting the aggregate to be resistant to the weather to afford protection to the coarse aggregate. But fine hair cracks occur in this outside coating almost invariably, permitting weather effects to reach the aggregate, for which reason the qualities necessary to resist weathering action are necessary also in the coarse aggregate. One of the important qualities of the aggregate, aside from durability, is the condition or

character of the surface for providing bond with the mortar or the cement paste. This was recognized in railroad circles many years ago and took the form of objection to gravel in comparison with broken stone because of the smoothness of the surfaces of the former—the requirement of crushed gravel is due to this cause; in the opinion of the author a tough durable coarse aggregate, but slightly porous, would be very satisfactory. Clean blast furnace slag free from sulphur after thorough seasoning would make an ideal coarse aggregate were it not for the difficulty encountered in mixing and placing.

Usually all too little attention is paid to the quality of the water used in the concrete mixture, even though it is now realized how great a part this constituent has in the quality of the resulting concrete.

Almost invariably the cement or aggregate is blamed for deteriorating concrete whereas in very many cases, possibly the majority, poor workmanship is probably responsible—with the chief offense failure of proper curing coupled with too much mixing water during installation. There is an all too frequent tendency, particularly on the part of company forces, to neglect this important after-care or curing, especially for sections placed at the end of a "job." This has been particularly true during the depression where a "gang" had to rush from a job as soon as completed to the next point. The basic difficulty, however, is the fact that the requisite experience and training is not expected of the concrete worker; a worker in lumber must be a qualified carpenter, one in steel an ironworker, in brick or stone a mason, but a common laborer is supposed to be competent to do concrete work.

Where these important factors have been neglected no maintenance coating is possibly available as a corrective. Even where every precaution has been taken, the duty imposed upon the coating is a difficult one. It must be effective both as a protection of the concrete against weathering or temperature effects and it must not detract from the architectural or decorative appearance. There are a number of products or methods on the market that meet one or the other requirement, but scarcely one that meets both for any length of time.

The author, in the examination of the structures herein described, and others, has been struck with the same puzzling feature that must strike all others who examine or inspect concrete structures that have been subjected to the weather. Structures that evidently have been honestly and efficiently designed and constructed will have sections, panels or members side by side sometimes in the same "pour" of concrete showing absolutely contrasting resistance to temperature and

weathering effects. Some of the illustrations herein will readily disclose this fact.

We must come to the conclusion that this is the result of our inability to control completely the uniformity of the output of the mixers; particularly is this true of the larger mixers and probably the larger the mixer the greater the variation in the portions of the delivery from the mixing drum even with full specified time of mix. This fact struck the author most forcefully when watching recently the unloading or discharge of a 4-yard truck mixer. There had been conscientious attention paid to all the requirements of proportioning and mixing (including time of mix and addition of water), yet the two ends of the discharge showed concretes of markedly different consistencies.

Yet, when we consider that a variation in the hundredths of one per cent of important constituents of steels and irons results in a marked difference in the characteristics and qualities of the product, it is surprising that even greater variations are not observed in concretes with their comparatively wide variations in grading and materials. Another important factor that is likely to be variable is the rate of absorption of the moisture of curing and the rate of evaporation after curing has ceased. This is particularly true for vertical surfaces where the only curing is all too frequently intermittant in the form of playing a hose on the surface at intervals (if at all). For, in the last analysis, the great contributory cause of the deterioration of concrete surfaces of structures that are properly designed and have not been subjected to unequal settlement, is invariably "shrinkage cracks" resulting from volume changes in setting and from subsequent changes in temperature. If proper curing can be continued for a period long enough to enable the concrete to attain a strength sufficient to withstand safely the stresses due to such volume changes there will be avoided the formation of these cracks which invite the entry of moisture and the alternate cycles of freezing and thawing which cause disintegration. Despite all precautions, however, fine hair shrinkage cracks occur for the protection of which surface coatings are required.

The condition of the structures about to be described is sufficient proof that under the climatic conditions to which they have been subjected concrete without some protection against climate is not "permanent." However, it indicates on the other hand that despite approximately a quarter of a century of severe exposure these structures have a long life ahead of them, particularly with a modest outlay for future protection. Neither space nor time permits of an exhaustive description of the condition of structures that have been in service 20 years and over. The descriptions will be confined to a

few outstanding structures or examples for special attention. The record of materials used is frequently not available nor that of the amount or method of curing.

EXAMINATION OF STRUCTURES

Tunkhannock Viaduct—The first structure to be described is the one that has been characterized as the "Eighth Wonder"—the Tunkhannock Viaduct, at Nicholson, Pa. It consists of ten semi-circular 180 ft. spans constructed in two 14 ft. ribs surmounted by open spandrel construction carrying a solid concrete deck for two ballasted tracks. These main spans are flanked by a 100 ft. semi-circular abutment span at either end completely buried in the stone fill of the approach embankments. The total length of the concrete structure is 2,375 ft. and its maximum height is 240 ft.—top of parapet above top of Tunkhannock Creek, and 300 ft. above rock foundation. All piers and abutments are founded on hard rock. Construction began in 1912 and the last of the 165,000 cu. yd. of concrete was poured toward the end of 1914. Provision was made for inspection of the arch ribs by means of openings left in the transverse spandrel walls over each rib, and rungs consisting of ¾ in. round rods imbedded in the concrete about 1 ft. on centers at the steep portions of the extrados as well as in the spandrel walls. It is therefore possible to climb around the arch ribs from one end of the viaduct to the other.

The materials used on this structure were sound broken dolomitic limestone from Syracuse, N. Y., for coarse aggregate, Hodgson sand from Netcong, N. J., as fine aggregate, and a then standard portland cement meeting in addition the autoclave test. The proportions were 1:2:4 for the floor system and 1:3:5 for the remainder of the structure (with derrick stone in the piers below springing line to form cyclopean masonry).

The condition of this structure last fall, at the time of the last inspection is perhaps best described by quoting the author's report after this inspection, supplemented by the photographs then taken:

> The condition of this structure has changed very little since the last detailed inspection of the viaduct. Except for the skewback and the ledges of the piers, the structure is in excellent condition, the only other points having any defects being the sidewalls of the refuge niches which, of course, have no structural value. There is slightly more disintegration at the skewbacks and the discoloration (from oil and creosote) of the concrete at the sides of the piers with minor cracks at this location between the arch ribs which mars the appearance of the structure but does not detract from its integrity. The balustrade, except in two or three spots in the entire length, is in very good condition. The walls of the refuge niches require immediate attention. I would recommend the use of emulsified asphalt with the addition of cement sprayed on before the emulsion has set so as to avoid the unsightly

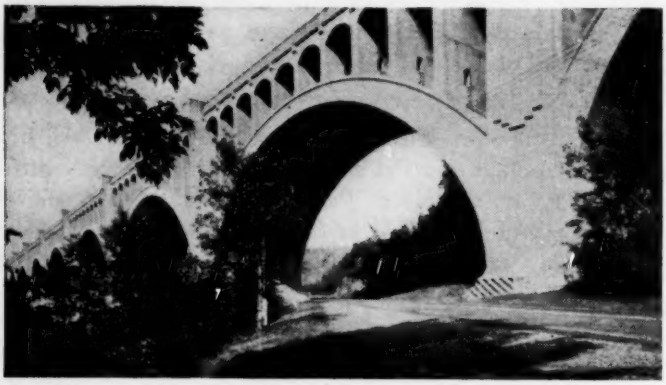


FIG. 3 AND 4—MARTIN'S CREEK VIADUCT—SEVEN 150-FOOT SPANS, TWO 100-FOOT SPANS AND TWO 50-FOOT SPANS—BUILT 1914.. THESE PICTURES MADE SEPTEMBER 1936

black appearance. I would also recommend the extension of the drainage system and the correction of the condition of the skewback as outlined four years ago.

The oversight in the design and construction of this viaduct was the failure to carry the drainage system down to the elevation where the drainage effluent could be disposed of without the possibility of its dripping upon the concrete or any part of the structure. A plan for remedying this condition by connecting all the drains and carrying the drainage down through a leader at each pier, together with electric heaters thermostatically controlled, was devised some years ago but has not yet been executed. This remedy will of course be much more expensive that would have been an initial installation—an ounce of prevention in this case would have been worth two pounds of cure. To date the only expenditures for maintenance or repair on this structure were the cost of sealing the cracks in the sidewalls of the refuge niches and that of a scaffold to provide for carrying a leader at one of the piers down to the ground level. The total cost of maintenance in 22 years has amounted to .07 of 1 per cent (of which 80 per cent is the cost of the scaffold), or at an annual rate of .003 per cent.

Martins Creek Viaduct—This structure, likewise on the Pennsylvania cut-off line, consists of seven 150-ft. spans, two 100-ft. spans and two 50-ft. spans, with top of parapet 150 ft. above the creek, and was constructed 1912 to 1914. All the piers and abutments are also founded on rock at various depths. Materials, except cement, came from the same sources as did those for the Tunkhannock Viaduct and the cement met the same specification requirements including that of the autoclave test. The present condition of the structure is also described in the report referred to above and reads:

The main structural members (piers, arches, spandrel walls and decks) are in excellent condition, no deterioration having occurred since the last inspection. The balustrade, however, has further deteriorated and the coating of . . (proprietary application) . . . is crazing similarly to the action of the untreated balustrade. The . . . (another proprietary coating) . . . application is in somewhat better condition than the . . . . . , but is also showing evidence of breakdown.

Both of these structures were built with expansion joints in decks and parapets and are almost entirely free from any cracks that might have occurred due to temperature change in the arch rings. The provision of such joints was the result of examination of the earlier viaducts (to be described) where such provision had not been made.

About 4½ years ago test applications of two proprietary coatings (referred to in the report quoted) were made on the balustrades over one of the spans, one on westbound side and one on the eastbound side. All in all, the cost of maintenance on this structure has amounted

FIG. 5—PAULINS KILL VIADUCT, HARRISBURG, N. J., BUILT 1909—FIVE 120-FOOT SPANS, TWO 100-FOOT SPANS

to approximately .06 of 1 per cent in 22 years. Possibly due to the reduced height or the provision of two drains in a line between the ribs, the effluent of the drains has not affected the piers to anywhere near the extent that is evident at the Tunkhannock. In this instance, very little expenditure would put this structure in perfect condition.

The two arch viaducts above described are on the cut-off line between Scranton and Binghamton which followed the construction of a cut-off line between Hopatcong, N. J., and Slateford, Pa., on which are two somewhat smaller arch viaducts built between 1908 and 1910.

Paulins Kill Viaduct—The most easterly of these is the Paulins Kill Viaduct near Hainesburg, N. J., completed in 1909. It consists of five 120-ft. and two 100-ft. semicircular spans with a height of 120 ft. from top of parapet to the stream. The piers and abutments were carried down to solid rock, while expansion joints were provided at every second pier only, with no provision elsewhere in the deck which is carried on open spandrel construction. The stone for the structure was obtained locally and is a sound limestone, the sand came from the Hodgson pit at Netcong, N. J., and the cement was a then current standard. The consistency of concrete in the structures on this cut-off was on the wet side. The mixtures were 1:2:4 for the decks and parapets and 1:3:5 for the remainder of the structure, with the inclusion of derrick stone for the piers. This structure, after 27 years of service, is also in excellent condition, the only flaws being occasional cracks in the parapets due to the movements of the arch resulting from changes in temperature as no expansion joints had been provided except at every second pier. These cracks do not show any disintegration, the concrete evidently thus far successfully resisting the weather. There are a number of spots on top of the parapet where pieces of chert have swelled and caused some "popping." The parapet is surmounted by a pipe railing supported by concrete piers;

FIG. 6—PAULINS KILL VIADUCT OF D. L. & W. RY. BUILT 1909—PICTURE MADE NOVEMBER 1936

a few of the latter show cracks due to the failure of the sections of pipe to slide in the openings provided for that purpose. This structure likewise is in such condition that a very small expenditure will put it in perfect condition. No expenditure for maintenance has been made on this structure in its 27 years of service except on rare occasions for opening drains that had become clogged.

Viaduct at Slateford—The fourth of the arch viaducts is near the west end of the cut-off line, across the Delaware River at Slateford, Pa., and consists of five 150-ft. and two 120-ft. elliptical arch spans and two 33-ft. semicircular spans which are about 65-ft. along the skew. Whereas all the other viaducts are right arches and on a tangent, this structure is at an angle with the river and has a rather sharp curve near one end The material used in this structure was very similar to that of the Paulins Kill Viaduct and conformed to the same specifications for proportioning, mixing and placing noted above. This structure was completed early in 1910, but does not present as pretty a picture as far as its condition is concerned as do the other

three viaducts though fully as effective in architectural appearance. However, when tested by a hammer in the area of unsightly crazing, the concrete was found to be sound, indicating that this was only surface condition. There are cracks in the parapet due to failure to provide intermediate expansion joints and the inefficiency of the deck waterproofing has caused water-wearing action including occasional spalling of the concrete. Since construction was completed, the barrels of the arches have been coated with a bituminous waterproofing coating which has constituted practically the only maintenance expense on this structure. The total cost of maintenance represents ⅔ of 1 per cent of the cost of the structure for 27 years, or an average of .025 per cent per annum. In this instance it would cost considerably more than in the case of the other viaducts to bring this structure up to par, but while the cost would not be prohibitive there would result considerable interference with railroad traffic. It may be interesting to note that an examination of the granite nosing of the piers shows apparently the same wear as the adjacent concrete.

Terminal Trainshed Support—The Buffalo structure mentioned at the outset is of an entirely different type than the viaducts described. It is a terminal trainshed support of the girderless flat slab type with drop panels—4-way system. The panels are 27 ft. square except at the tapering ends of the viaduct where the structure narrows down from 154-ft. width (including 9½-ft. cantilever ends) accommodating 7 tracks and 4 platforms to 45 ft. for 3 tracks without platforms. The structure is founded on piles throughout with individual concrete footings for each column except at converging bents. The materials here used were Niagara River gravel and sand and standard portland cement (with the addition of the autoclave test). The proportions were 1:2:4 for columns, drop panels, slabs, parapets and balustrade, an 1:3:5 for the mass footings and gravity retaining walls, the mixtures being kept fairly dry. The curing called for in these specifications was to continue for seven days "by thoroughly drenching with water 3 or 4 times a day." The construction was completed in 1916. An examination of this structure finds it in excellent condition despite minor cracks in the cantilever slab which occurred because of the slightly unequal settlement of the foundations. These cracks have shown no sign of causing disintegration of the concrete and have not resulted in any expenditure for maintenance thus far in the 20 years of its life.

South Orange Avenue Viaduct—A structure of similar type about 15 miles from the eastern terminus of the railroad, built 1914 and 1915, is the South Orange Avenue Viaduct at South Orange, N. J. This

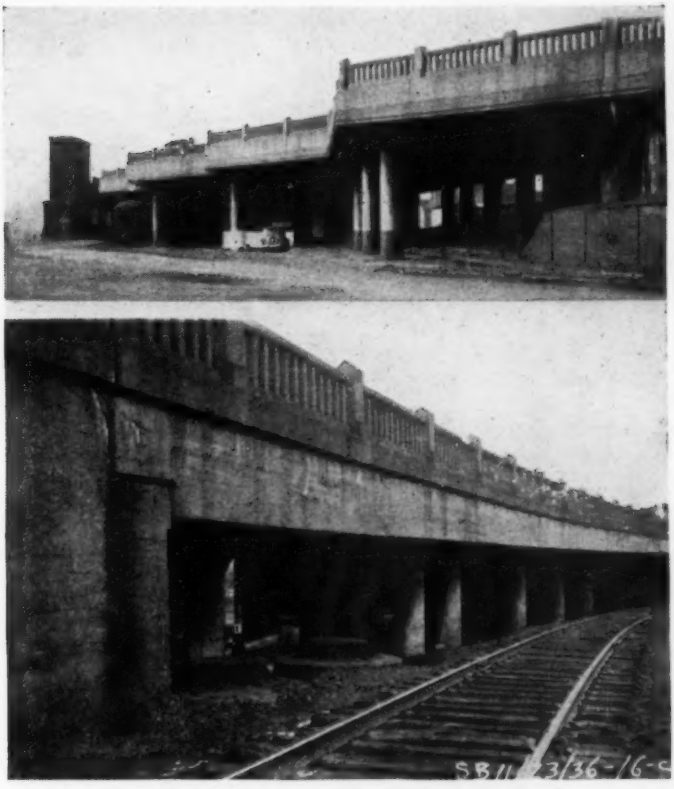


FIG. 7 AND 8—BUFFALO TERMINAL, GIRDERLESS FLAT SLAB VIADUCT, 1100 FEET LONG, 27-FOOT SQUARE PANELS, BUILT 1914-15. PHOTOGRAPHS MADE NOVEMBER 1936

is a flat slab viaduct about 450 ft. long and 79 ft. at its greatest width, accommodating 3 tracks and 2 platforms. The main panels are 20 ft. by 18 ft. 9 in. with two 30-ft. spans center to center of columns, over the avenue, for which section the thickness of the slab is increased to 2 ft. 2 in. from 1 ft. 8 in. elsewhere, between the drop panels. The concrete in this structure was placed in unusually large sections between construction joints which were only formed transversely, the longest section being at the South Orange Avenue end. There was provided no expansion joint in its whole length. The material used was crushed trap rock for coarse aggregate, Hodgson sand from Netcong, N. J., and autoclave cement. The proportions used for the columns, slabs, parapets, stairways and platforms were 1 cement to 2 sand to 4 stone, while a 1:3:5 mix was used for abutments, column footings and retain-

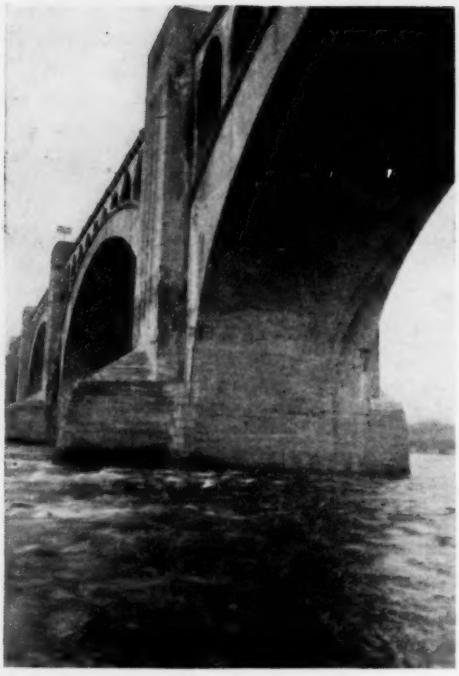


FIG. 9 AND 10—DELAWARE RIVER BRIDGE BUILT BY D. L. & W. RY. 1909—ELIPTICAL ARCHES; FIVE 150-FOOT SPANS, TWO 100 FOOT SPANS, TWO 33-FOOT SPANS—65 FEET ON SKEW

ing wall with an amount of water giving a consistency on the wet side. The columns and faces of abutments and parapets were bush-hammered. This structure, as well as the others previously described, were waterproofed with a 2-ply membrane (3-ply at the Tunkhannock) of asphalt saturated fabric and a protection coat to prevent puncture of the membrane by the ballast.

FIG. 11—MOUNTAIN STATION BRIDGE (1914)—56 FOOT SPAN, MONTROSE AVENUE, SOUTH ORANGE, N. J.

An examination of this structure showed it to be in very good condition in the main except in the vicinity of the stairways at the east end. The facia stair string at the south (eastbound) stairway is cracked near the support due to the fact that the sliding joint which was provided for changes of temperature failed to permit the movement anticipated. The same fault caused cracking of the stair string of the island platform stairway which in turn caused damage to the balustrade over it. The correction of this condition will be described under "Repairs." This repair was carried out about 3 years ago when the structure had been in service almost 20 years. The only other expenditure for maintenance of the structure was that for correction of the waterproofing, the protection for which at the platform had bent away from the curb and carried the membrane with it, thus permitting the water to enter and find its ways to the top of the slab. This same condition now exists at another section of the viaduct and will be corrected. Altogether, the cost of repairs and maintenance for the 22 years of service of this structure forms a percentage of its cost not much greater than that noted above for the arch viaducts. Here again a very small expenditure for repairs will bring this structure up to first-class condition.

Smaller structures—Of the numerous smaller structures that were built in the period under discussion, a few are selected for illustration without detailed description as further illustration of the ability of concrete structures to withstand the weathering effects of this climate and to indicate that we cannot rely on *entire* elimination of maintenance in concrete structures. While many of the faults are struc-

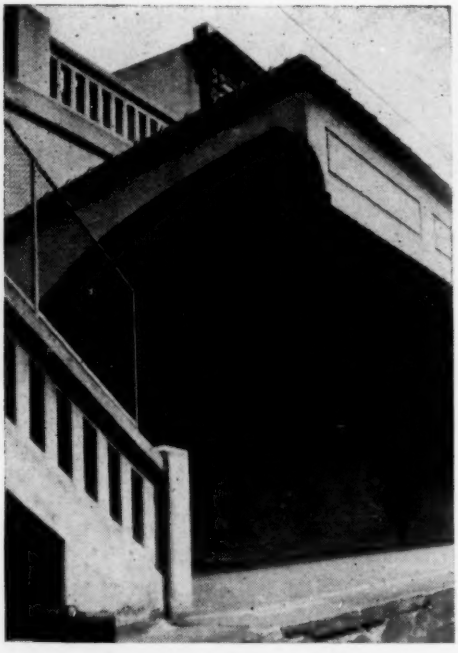


FIG. 12—WATSESSING AVENUE, BLOOMFIELD, N. J.—PLATFORM CANOPY AND APRON OF REINFORCED CONCRETE BUILT 1909-10—PHOTOGRAPH SEPTEMBER 1936

tural they yet are inherent in concrete and are due to the nature of concrete design with its reliance on continuity which is at the same time its great strength as a resourceful material and its great weakness in subjecting it to cracking under any condition not perfectly suitable or not entirely foreseen; this is also true in connection with volume changes due to shrinkage and temperature variations.

The structures above referred to are the overhead 56-ft. elliptical arch bridge at Mountain Station, built in 1913, which is in excellent condition; Third Street, South Orange, N. J., a slab undercrossing over 4 spans, built in 1914, also in excellent condition; Lafayette Street, Chatham, N. J., a 46 ft. segmental arch undercrossing built in 1913, which is in very good condition except for points on top of the wingwall where the disintegration shows evidence of water gain; a 47-ft. span arch undercrossing at Tobyhanna, Pa., built in 1911, shows the same conditions; two bridges west of Dalton, Pa., one an arch undercrossing, 24-ft. span, in almost perfect condition; while the arch overcrossing of 90-ft. skew span just west of it has its beauty

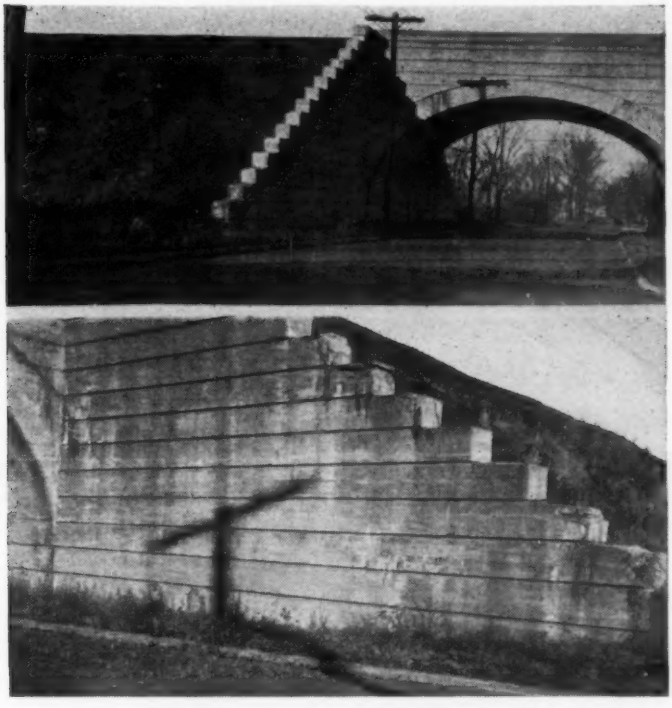


FIG. 13 AND 14—LAFAYETTE AVENUE ARCH UNDERCROSSING, CHATHAM, N. J., BUILT 1914; PHOTOGRAPHS NOVEMBER 1936

marred by evidence of over-wet mix, though both structures were installed by the same contractor in 1913, using the same materials with same inspection under the same specification. The result can only be attributed to an attempt to obtain too smooth surfaces and details in the more ornamental structure by increasing workability by means of additional water. A change of material to local gravel in an arch bridge at New Milford, Pa., built in 1913, resulted in a structure that, while perfectly safe, shows disintegration due to that cause. The first precast slab bridge (undercrossing—two 20-ft. spans) installed at Moscow, Pa., in 1911, shows but small effects of 25 years of service and that due to lack of waterproofing and rather wet placement of concrete for sections poured in place. The 4-span slab bridge at Madison Avenue, Madison N. J., is in excellent condition except for minor cracks due to foundation conditions and slight evidences of laitance at horizontal construction joints.

FIG. 15—ARCH UNDERCROSSING, TOBYHANNA, PA., BUILT 1909—PHOTOGRAPH NOVEMBER 1936

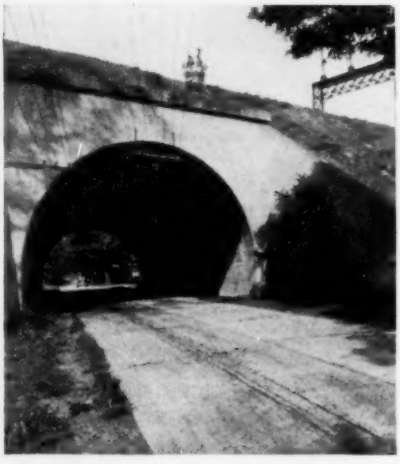


FIG. 16—SEMICIRCULAR UNDERCROSSING WEST OF DALTON, PA., BUILT 1913

No maintenance costs have been incurred on any of these structures to date and comparatively small expenditure would put them all in first-class condition.

Where structures are small and not subjected to undue stresses, they have withstood the ravages of time almost perfectly without any maintenance whatever. All the reinforced concrete stations on both

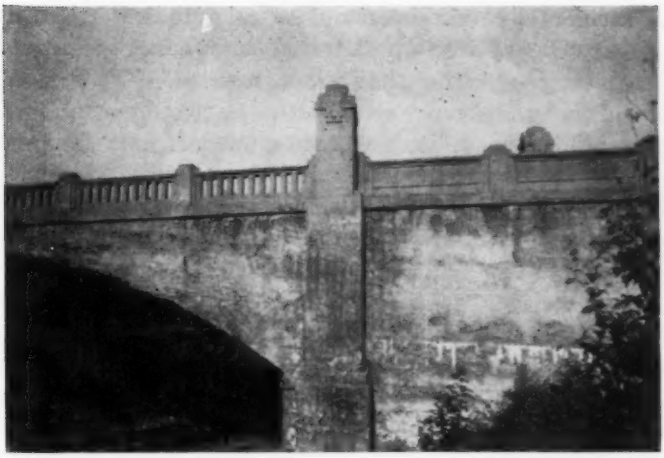


FIG. 17—FULLER'S ARCH, OVERHEAD CROSSING WEST OF DALTON, PA., (NEAR FIG. 16)—BUILT 1913—PHOTOGRAPH SEPTEMBER 1936

"cut-off" lines built between 1908 and 1914 are in excellent condition —the Nicholson, Pa., station(1914) and the one at Bloomfield, N. J. (1911) are good examples as is also the Signal Tower at Tobyhanna, Pa. (1911). At the Watsessing Avenue Station at Bloomfield, N. J., the thin section (3 in. thick) of concrete apron reinforced with mesh metal, built in 1911, hanging 4 ft. deep from the concrete canopy, has stood up remarkably well, though subjected to locomotive exhaust for many years. It is in practically perfect condition after 25 years' service. The balustrade at the same location is in fine condition except for one post.

REPAIR WORK

The second section of this paper has for its scope the description of some of the repair work found necessary of installation on concrete structures in various sections of the railroad.

These repairs may be classified in accordance with their depth and surface area into (a) patching, (b) coating, (c) encasement, (d) replacement of a member or part of a member. Both (a) and (b) are assumed to refer to repairs which involve a depth of ½ in. to a maximum of 2 in., the difference between them being in surface area, while (c) refers to coatings having a thickness greater than 2 in.; (d) obviously is self-explanatory.

(a) *Patching*—The greatest amount of patching has been in connection with repairs of station and freight platforms, floors and stair

treads. Incidentally, our experience seems to indicate that platforms not subject to heavy iron-wheel traffic show best resistance to wear when broom-finished rather than when troweled. The first attempts in repairs of such patches are with mortar or concrete containing small size aggregate. The difficulties encountered in such work are the formation of new shrinkage cracks and the inability to keep the area out of service long enough to permit proper curing for the patch to attain its strength in bond at the junction of the old and new concrete. Quick-setting cements and high-early strength cements, while reducing, did not eliminate the trouble. Many proprietary products have been tried with almost uniformly unsatisfactory results. The one exception is a product (proprietary) which when used in concrete results in a remarkably small amount of shrinkage in locations not exposed to the open weather. It is very effective even when applied to the chipped nosing of a stair-tread; but this material likewise failed to produce satisfactory results in locations of severe exposure. Concrete made with a bituminous emulsion has also given satisfactory results in patching, but on the other hand has not been as resistant to wear aginst traffic where the latter is at all heavy. Aside from the following requirements fundamental to all repairs, patches, in the observation of the author, have been most nearly successful when sharp distinct straight line joints are made separating the new area from the old, similar to a construction joint.

1. All loose material, rust or scale from reinforcement must be completely removed.
2. The exposed surface (roughened) must be thoroughly cleaned with sand blast if necessary.
3. The surface to which the patch is to be applied must be saturated with clean water just prior to the application of the patch.
4. The first coat must be applied with force to avoid the formation of dust films.
5. Thorough curing is absolutely necessary without disturbance of the applied material.

(*b*) *Coatings*—Under this head (not mentioned above) are also included all the proprietary applications or paints that are on the market, of which we have tested a great many in our Scranton testing laboratory and several with test installations. It is not the province of this paper to discuss these products. However, the test applications (of those that seemed to have attractive possibilities) have not been successful in resisting the weathering of the severe climatic conditions to which they have been subjected. In their application the precaution was taken to remove all unsound concrete, clean by means of sand blasting, and to cover the exposed surface with the sealing material and then follow with final coat or coats. Under this head would

also be included the bituminous coatings, emulsified or cut backs, which, while frequently effective, are unsightly.

Our greatest success in such repair work has been by the use of pneumatically projected mortar for which process the American Railway Engineering Association has adopted the term "shotcrete" as expressive of the procedure. The best example of such work on our lines is the coating of retaining walls at Buffalo, N. Y., which had been installed in 1906-7 and because of unsound coarse aggregate had to be restored in 1924. This work has withstood weathering very well in the period since the installation. Due to its success at this location, the process was used for many smaller areas on the Buffalo Division. Equal success was had with this type of coating for tunnel lining repair in the Kingsland (N. J.) tunnel near the eastern end of the rail road. The application in all cases was made in layers of ½ in. to ¾ in. thick and where the thickness of coating was appreciable, dowels and reinforcement were used. Dowels are very effective if properly imbedded in the old concrete because when shrinkage of the new concrete occurs with its tendency to part from the old concrete the dowels are put in tension—their proper function as concrete reinforcement. Aside from cleanliness of surface to which the coating is to be applied, other precautions have to be taken with this type of work. These are incorporated in the specifications covering this and other types of repair work (appended). The specifications are similar to those adopted by the American Railway Engineering Association. They are a basis for proposed restoration of retaining walls in what is known as "Roseville Cut," at Newark, N. J., in service more than 35 years and still sound, but unsightly at numerous points. There is but one drawback in the use of "shotcrete" and it is that it is not amenable to troweling to obtain a smooth finish.

(*c*) *Encasement*—This type of repair is limited to coatings more than 2 in. thick for shotcrete and more than 4 in. for concrete in forms and is used where deterioration or failure is general for any part of a structure. An example of this type of repair is that of a counterforted reinforced concrete retaining wall in Oswego, N. Y., which had been erected in 1905. After a number of years in service the face was found to be out of plumb and closer examination upon removal of fill near the top disclosed the fact that the main counterfort reinforcement had not been extended sufficiently into the face wall, causing the two to part at the top and gradually to shear to greater depth. This condition became progressively worse until in 1924 it became imperative to correct it. The remedy was an encasement in accordance with the details shown (Fig. 18) A face was placed in front of

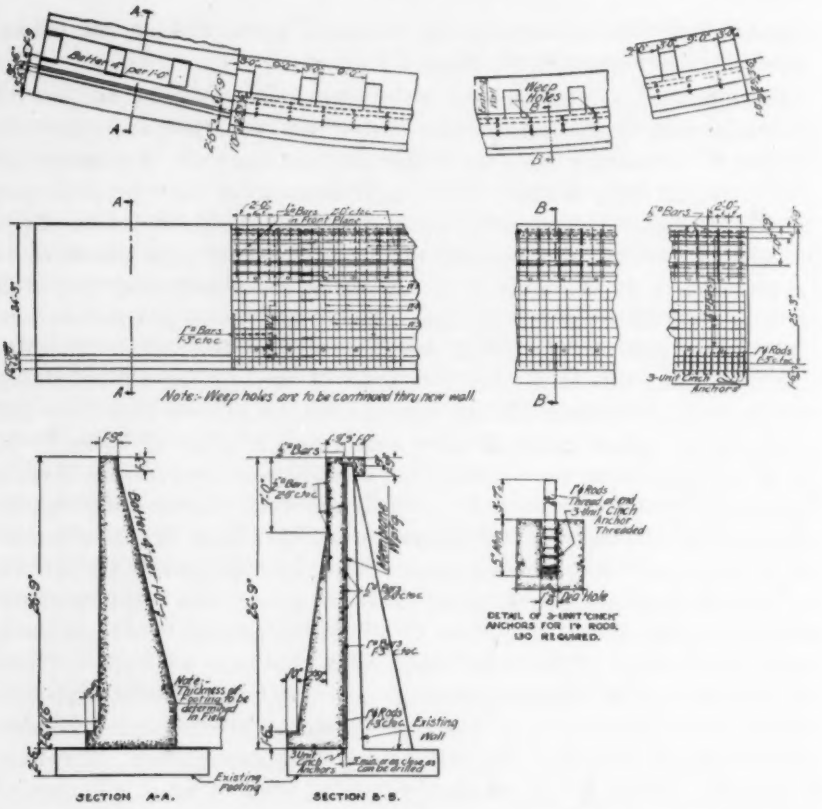

Note:- Weep holes are to be continued thru new wall
Membrane Waterproofing
Existing Wall
Existing Footing
SECTION A-A.
SECTION B-B.
DETAIL OF 3-UNIT "CINCH" ANCHORS FOR 1¼ RODS, 130 REQUIRED.

FIG. 18—RETAINING WALL REPAIR, OSWEGO. N. Y.

the face wall of sufficient section to resist the pressure of the filling material and was securely anchored to the footing to insure action of the new wall with the old footing by means of cinch anchors. This work was, of course, a form job and was installed without any difficulty and has served satisfactorily since. Strictly speaking, this was an addition to the wall rather than a Simon-pure repair and, of course required a special analysis and design of the combined structure. A blanket of membrane waterproofing between the old and the new face walls was provided as insurance against entrance of water between the walls.

Another example of an encasement repair is that at South Orange, N. J., in connection with the end stairway in which the "strings" had developed a crack near the supporting wall (referred to earlier in the paper). To shorten the span of the stair string to a point

beyond the shear crack, a wall 12 in. thick (a column under each string was the original plan) was placed in front of the platform wall for the width of the stairway and carried up to the underside of the stair strings. Dowels and reinforcement were provided in a plan drawn for the repair. This work was done more than 3 years ago and has apparently remedied the condition.

(*d*) *Replacement*—An example of repair under classification (d) is that of the replacement of a portion of a pier which acts as the support of the sliding end of a structural steel girder span in a viaduct which was built in 1915 in a grade elimination of Park Street, Dunmore, Pa. The remainder of the spans are of reinforced concrete—a main arch span with beam and slab construction for approaches. This repair was made necessary because of the failure to grout-in the tubes that were left for additional bolts in the bridge seat. Water entered these openings and froze during each winter, then thawed and when a sufficient number of these cycles of freezing and thawing had taken place, cracks which developed in the pier caused further disintegration so that by 1926 it was necessary to make the repairs in accordance with the plan shown. (Fig. 19 and 20) The procedure was as follows:

Falsework was placed near the support to relieve the pier of the girder reaction and the concrete of the pier removed until sound concrete was reached (to the lines indicated on the plan), dowels were placed into the sound concrete both horizontally and vertically after which the concrete surfaces were thoroughly roughened, cleaned and reinforcement placed within the forms. The concrete was then placed and after proper curing put in service again and has successfully resumed its function as a bearing pier for the girder with no sign of any deterioration in the 10 years since this installation.

It may be interesting to note that the method of pneumatically projected mortar coating has been very successfully applied in the repair of numerous stone masonry abutments on the Buffalo or western Division of the Railroad.

Of the 45 structures examined or inspected in connection with this paper, there is not one where really serious disintegration has occurred even though the structures selected for inspection were those built in 1916 or earlier. There are some that have portions that are unsightly and which will have to be rehabilitated sooner or later, but in the main they have given a good account of themselves.

CONCLUSIONS

The conclusions reached by the author from his experience with concrete structures, supplemented by the special inspections of the structures above described and referred to, immediately preceding the writing of this paper, are as follows:

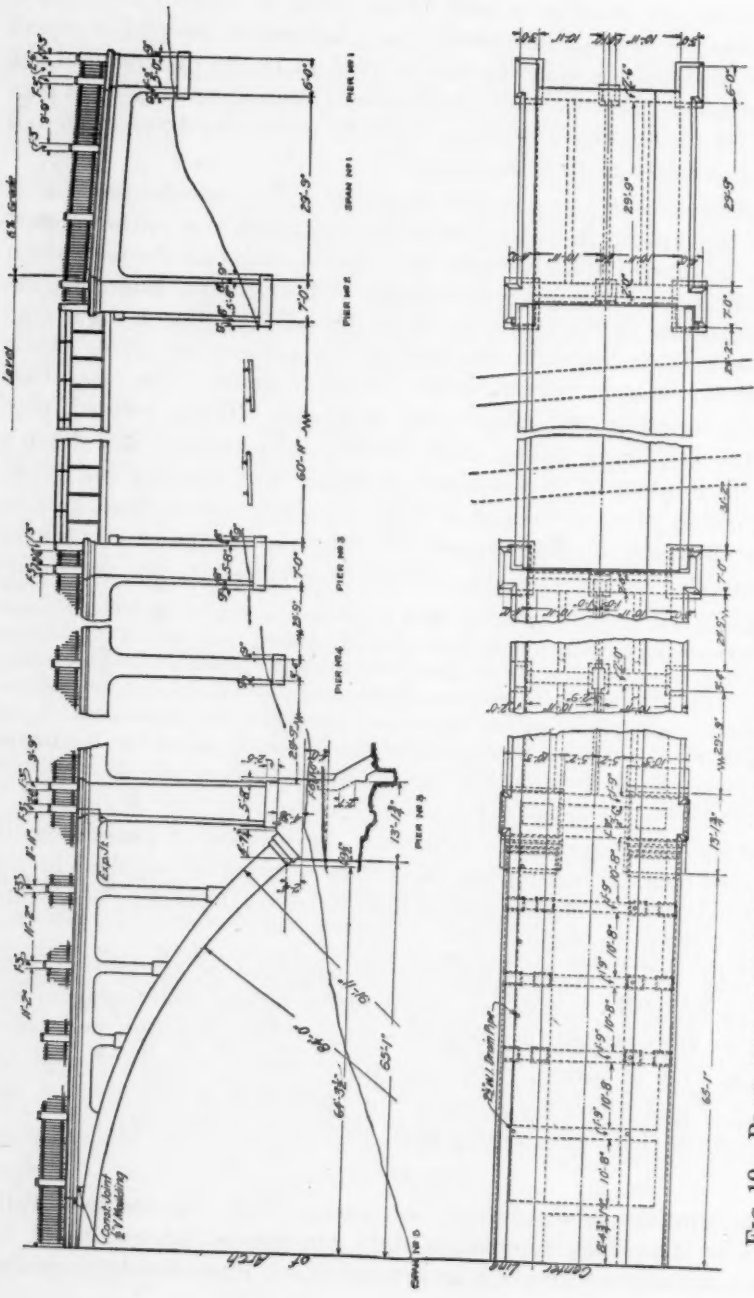


FIG 19—PLAN AND ELEVATION OF SOUTH HALF PARK STREET BRIDGE, DUNMORE, PA.—ENCASEMENT REPAIR (SEE FIG. 20)

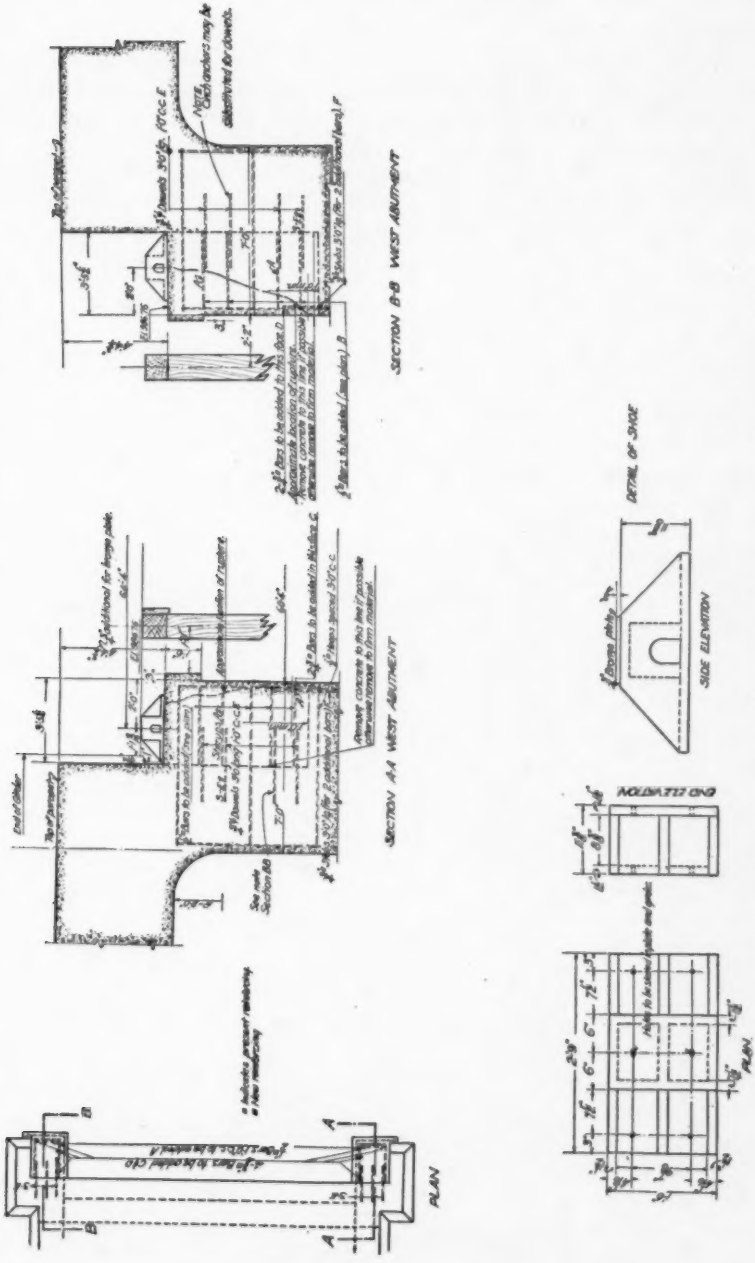


FIG 20—DETAILS OF DUNMORE, PA., REPAIRS (SEE FIG. 19)

Causes of cracks and deterioration in concrete structures may be attributed to:

(*a*) *Structural*

1. Unequal settlement of foundation of the structure—at times resulting from failure to separate parts of structures resting on soils of different bearing power.
2. Failure to provide distinct joints at changes in cross-section.
3. Failure to provide against distortions transferred by main members which are subject to change in configuration as a result of temperature changes—(such as spandrel construction surmounting arches).
4. Failure of joints (chiefly sliding joints) to act as designed.
5. Failure of drainage system—included in this item is failure to anticipate accumulation of water in areas not intended to carry water such as platforms or cinders supported by slabs of structures.
6. Failure anywhere to provide for volume changes due to shrinkage or to temperature changes.

(*b*) *Installation—Material or Methods*

1. Unsound or faulty materials—any of the ingredients in the concrete.
2. Excess of water in the mix.
3. Water gain in placement.
4. Segregation in placement.
5. Closeness of reinforcement to exposed surface. It may be noted that remarkably few instances of this error were found in the structures examined.
6. Formation of laitance at joints or temporary stoppage of concrete pour.
7. Insufficient curing.

In making repairs of any kind, the following precautions are necessary:

1. If at all possible, correction of condition causing the failure, particularly in the case of faulty drainage.
2. Concrete to which repair is to be applied must be of undoubted soundness.
3. Surface to which repair is applied must be perfectly clean (sand blasted preferred) and thoroughly roughened.
4. Bonding to old concrete is of utmost importance—the additional bonding reinforcement of dowels should be provided where possible.
5. Where sufficient thickness is available, reinforcement of mesh or rod mats should be installed.
6. Where feasible, provide clean cut joints at surface between new and old concrete.
7. Curing of patch or other repair is of utmost importance to minimize danger of formationof shrinkage cracks.
8. Strict adherence to all parts of specifications for repairs is essential.

APPENDIX—SPECIFICATIONS FOR CONCRETE REPAIR WORK

1. Anchorage

Dowels and expansion bolts shall be set after the bonding surface is exposed. Any space in the old concrete around dowels or anchor bolts shall be filled with cement mortar rammed into place or by grouting. The concrete surrounding dowel holes shall be kept thoroughly wet for a period of at least one hour prior to packing or grouting. Anchors so packed shall be set at least two days prior to attachment of

reinforcement to them or to placement of concrete, and shall not be disturbed during that time.

(*a*) *Attachment of Reinforcement*—Dowels, approved expansion bolts, or other anchorage shall be used where necessary to hold the reinforcement in place.

(*b*) *Provision for Bond*—Where it is impracticable to secure a good natural bond between the new and old concrete or where tension or shear at the bonding surface will be a principal stress, the integrity of the action of the new concrete with the old shall be assured by dowels and/or expansion bolts of sufficient size, number and embedment to develop the strength of the new concrete in combination with its reinforcement, if any.

2. Preparation of Bonding Surface

(a) The bonding surface shall be rough and clean. Loose particles, dust and dirt shall be removed by vigorous brushing with wire brushes, sand blasting or otherwise, prior to application of new material. Oil or film of any sort that may reduce the bond of the new material to the old concrete will not be permitted.

(b) The bonding surface shall be maintained constantly wet for a minimum of one hour prior to application of new material. After the wetting period sufficient time shall be allowed to elapse, or special means used, to remove all surface wetness and to produce a damp surface that is slightly absorptive. The fresh material shall be applied when this condition is attained. In no case shall fresh material be applied to a dry surface.

3. Application of New Concrete

(*a*) *Bonding Coat (Hand Work)*—The first or bonding coat shall be composed of one part cement to one part of sand. The mortar for bonding coats shall be of plastering consistency and shall have been mixed from one-half to three hours (depending on temperature and humidity) prior to using, but shall be plastic at the time of application. Water shall not be added to retemper the bonding coat mortar. The bonding coat shall be applied to a thickness of approximately ¼ inch immediately prior to placement of the body of the new concrete. It shall be projected or splattered against the prepared surface with force and shall not be troweled, screened, or disturbed until the next layer of new concrete is applied.

(*b*) *Built-up Patch—Overhead, Vertical or Steep Surface.* Successive layers of concrete shall be applied if practicable while the preceding one, including the bonding coat, is still plastic, and until the requisite amount of concrete is applied. If necessary to prevent sloughing of an already applied layer, it may be allowed to harden in place until it has acquired sufficient strength to hold the next layer, but in this event the next layer shall be applied as for a bonding coat. The cavity may be overfilled and brought to approximate surface by hammering or pushing on a board in contact with, and extending beyond the patch. Excess concrete shall be carefully removed with a sharp tool and the patch brought to the required surface by patting with a wood float or trowel. Heavy troweling or working of the finished surface will not be permitted, until the concrete has acquired sufficient stiffness to prevent disturbance of its original position below the surface torweled. All troweling and/or floating of the plastic surface shall be lightly done so as to limit its effect as far as possible to the surface.

(*c*) *Built-up Patch—Horizontal Top Surface*—After the bonding coat, the concrete shall be rammed into place until the cavity is overfilled. The patch shall then be further consolidated by hammering on a board in contact with but extending beyond the patch. This shall be repeated prior to initial set of the cement. Excess concrete

shall be carefully removed with a sharp tool so as not to drag the surface and the final finishing accomplished by patting and light troweling.

(*d*) *Packing in Forms*—Where restoration or encasement is accomplished by ramming the new concrete in between forms and the old surface, the forms shall have sufficient strength to withstand the pressure of the new concrete without yielding appreciably. If possible, the new concrete shall be placed in layers, each rammed solidly in place while the preceding layer is plastic. In any case the new concrete shall completely fill the space provided and present a surface identical in location with the original. Vibration of the forms in lieu of ramming is subject to approval of the Engineer. Vibrators shall preferably deliver not less than 3000 impulses per minute.

3A. Shotcrete Work

Shotcrete shall be a mixture of one part portland cement and approximately three parts sand free from particles ¼ in. size and over. The amount of sand shall be based on dry, loose measurement but shall have not more than 8 per cent by weight of moisture when mixed with the cement. The sand and cement shall be thoroughly mixed before being put into the hopper of the cement gun.

The air pressure in the shotcrete gun shall be maintained uniform and not less than 35 p.s.i., while placing the mixed materials. This pressure shall be increased for horizontal delivery distances exceeding 100 ft. and for vertical distances exceeding 25 ft. The water pressure shall be not less than 25 p.s.i. more than the air pressure. The nozzle of the gun shall be directed as nearly as possible at right angles to, and shall be held at 2 ft. to 4 ft. from the working surface.

A sufficient number of coats shall be applied to obtain the required thickness but not less than two coats shall be placed. The thickness of each coat shall be such that it will neither slough nor decrease the bond with the preceding coat. At least two hours shall elapse between the placement of succeeding coats.

Shooting strips shall be used to obtain true lines, corners and surfaces, but the final or flash coat shall not be troweled or worked in any manner.

4. Curing and Protection

The surface of all new concrete shall be kept thoroughly wet for a period of at least seven days beginning immediately after placement except where high early strength concrete is used in which case the curing period may be shortened as determined by the Engineer to that which will produce equivalent strength.

Repair work shall not be done in freezing weather unless properly protected nor until the old structure is free from frost. The new concrete shall have a temperature when place of not less than 5°0 F., nor more than 120° F., and shall be kept at a temperature not lower than 50° F., for not less than 72 hours after placing, or until the concrete has thoroughly hardened.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by April 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for May-June, 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . JANUARY-FEBRUARY 1937

Cement and Concrete Studies on the Passamaquoddy Tidal Power Project*

By Hugh J. Casey†

In connection with the Passamaquoddy Tidal Power project, the design and construction of which were undertaken by the Corps of Engineers in May 1935 as a major work relief project under an allotment of funds from the Emergency Relief Appropriation of 1935, extensive cement and concrete studies were made.

DESCRIPTION OF PROJECT

Although small tidal power mills have been in operation in this country and in Europe for centuries, the Passamaquoddy project is the first large-scale tidal power project ever to have been undertaken. The Quoddy project was to harness the power in the high Fundy tides which prevail in the vicinity of Eastport, ranging from apogean neap tides of less than 9 feet to extreme perigean spring tides of almost 27 feet with mean tidal range of 18.1 feet. As may be generally known, it was the plan of Dexter P. Cooper, the original proponent of the project, to enclose Cobscook Bay and Passamaquoddy Bay, arms of the Bay of Fundy, from the latter bay by a series of dams, gate-structures and navigation locks. Between Passamaquoddy and Cobscook Bays a series of dams and a power house were to be constructed. (Fig. 1).

The plan of operation was that at and near high tide the gates to Passamaquoddy Bay would be opened and the Bay filled to near high tide levels. As the tide receded the gates would be closed. Conversely at and near low tide, the gates to Cobscook Bay would be opened and the Bay drawn down to near low tide elevation. Thus there would be a head produced between the two pools, available, through the interconnecting power house, for the production of energy.

*Received by the Institute January 11, 1937, for presentation 33rd Annual Convention.
†Captain Corps of Engineers, U. S. Army, Boston.

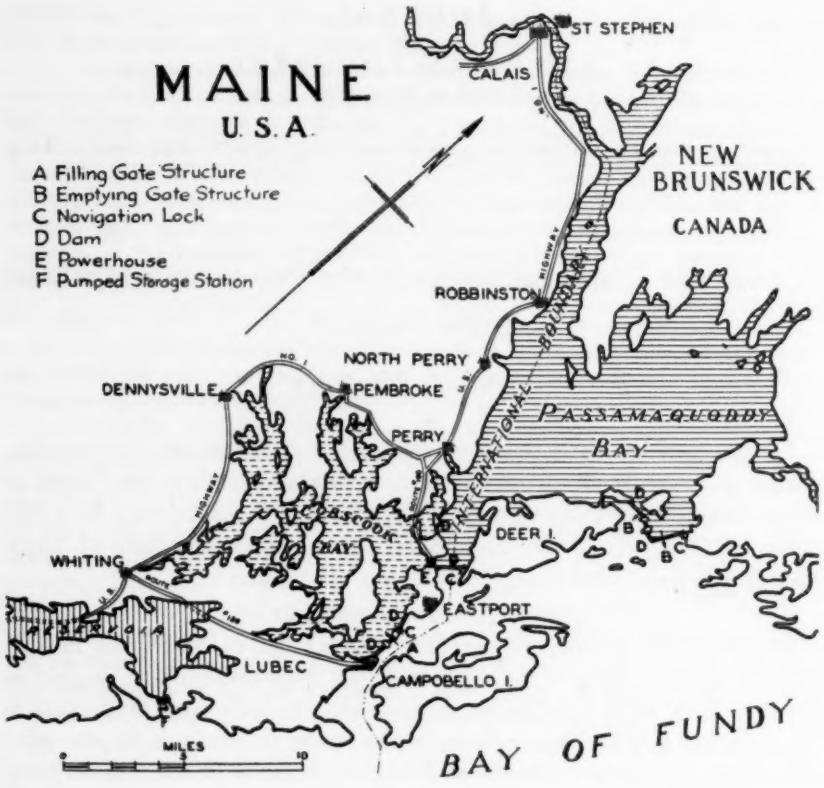


FIG. 1

The project authorized in 1935 was for the construction of only one step of the two-basin plan. It provided for the development only of Cobscook Bay, a 37 sq. mile (at high tide) estuary lying entirely in American waters. (Passamaquoddy Bay lies in Canada and its ultimate inclusion in a two-basin development will require the cooperation of Canada.) The plan of operation was to drain Cobscook Bay to low tide levels and when the outer tide rose some 5½ feet higher than the interior tailwater basin level, to open the power house, generating power on the rising tide to high tide level and on the falling tide until the head was reduced again to near 5½ feet. The power house would then be shut down due to insufficient head. As the tide fell further to the level of the interior basin, the gates would be opened and the "tailwater" in the basin permitted to drain to near low tide levels and the cycle resumed.

Whereas the two-basin development would provide some constant power, the single-basin development would require supplemental priming means to develop any firm power. For of the 12 hr. and 25 min. tidal cycle, power could be produced in varying amounts for periods of approximately 7¼ hours only. Cooper proposed a pumped storage reservoir at Haycock Harbor, about 16 miles distant, where surplus electric energy produced at and near high tide would be stored in the form of potential hydraulic energy by pumping sea water up into the artificially created high-head storage reservoir, to be drawn on for regeneration during shut-down periods at the tidal station.

CONCRETE PROBLEM

Under this program, as originally contemplated, providing 30,000 KW firm output and including a 10-unit 150,000 KW power station, with navigation lock, a commensurate gate structure and a large pumped storage development, a total of approximately 800,000 cu. yd. of concrete would have been required. This concrete would have been subject in varying degree to severe treatment from salt water action, intense freezing and extensive alternate wetting and drying. The quantities and conditions were such as to warrant an extensive concrete research program.

MODIFIED PROJECT

Our engineering investigations, including thorough foundation investigations, soils mechanics studies, model tests of turbines, scroll cases and draft tubes, complete hydraulic and power studies, and detailed designs and estimates, indicated the necessity of a material revision upward of the cost estimates of the project's initial promoter. They also disclosed that a better and cheaper project could be attained by (1) development of Cobscook Bay as a high-level rather than low-level basin with reversed operation from that originally planned, and (2) elimination of the pumped storage feature, and substitution therefor, for primary power, of power interchange with existing utilities or the provision of a thermal or hydro-electric auxiliary stand-by plant. With these changes it was also found that an equivalent energy output to that contemplated in the initial development (30,000 KW prime) could be attained by a reduction in size of the main tidal power and gate structure installation. Our total concrete requirements were thereby reduced to approximately 260,000 cu. yd. Inasmuch as the extensive concrete research program had, however, already been initiated on the basis of the larger project, and inasmuch as it was felt that these studies would prove to be of general value to the Corps of Engineers, to the engineering profession and general public, the pro-

gram was continued until curtailment of activity on the project due to the action of Congress in not authorizing the project and appropriating no additional funds for its continuation. Certain phases of this investigation including maintenance and periodic inspection of 43 different concrete test columns now exposed at Eastport to salt water action at half tide elevation will, however, be continued indefinitely.

The conditions affecting Quoddy concrete are, as previously stated, most severe. Initially, this concrete is subject to salt water action, commonly recognized as injurious to normal concrete; secondly, due to the extreme tidal range existing at Eastport including a mean range slightly in excess of 18 ft. with maximum range approaching 27 ft., a large area of concrete is exposed to alternate wetting and drying twice each lunar day (24 hrs. 50 min.). Water under varying pressure is thus forced into the pores of the concrete at high tides and subsequently exposed at low tide to atmospheric effects; thirdly, due to the latitude at Eastport, concrete is subject to severe freezing temperatures.

BASIC ESSENTIALS

It must be recognized initially that there is no single panacea or cure-all for these conditions. It was not expected that some special cement or admixture or process would be found or developed which of itself would ensure a perfect concrete resistant to all these conditions.

The simple elements of the problem are generally as follows: A dense and impermeable masonry mass must be formed of durable stone and sand aggregates strongly cemented together, with the mass and its constituents unaffected by the physical changes resulting from heat and cold and from wetting and drying or by the chemical attack of salt water.

This dictates first that the *physical* structure of the concrete be sound and free from honeycombing, pores or tiny fissures which would furnish incipient "paths of deterioration" from freezing or incurrence of salt water. This requires: (a) selection of sound, dense, clean and well-graded coarse and fine aggregates; (b) limitation of water content to the minimum required for proper workability, as any water over and above that necessary for the hydration of the cement will leave pores corresponding to the physical space occupied by the water prior to its evaporation; (c) rigid control of mixing and placing, including vibration, to ensure a uniform mix of dense compaction, without segregation, of the concrete; (d) choice of a cement subject to minimum physical change in expansion or contraction in the process of setting. Inasmuch as the rate and extent of expansion and contraction are

functions not only of the size of pour but also of the chemical composition and fineness of the cement, which affect its heat of hydration, control must be exercised over these features to prevent temperature cracks and fissures; (e) protection of the setting concrete by proper curing, including controlled cooling and protection from severe cold, to avoid insofar as possible excessive temperature changes within the mass, particularly during the early period when tensile and compressive strengths are still low.

The next major requirement is to ensure that the dense and impermeable masonry mass previously defined is protected from *chemical* disintegration. For this purpose, (a) aggregate must be sound and durable and not subject to disintegration from the chemical attack of sea water, and (b) similarly, but of even more importance, the cement, the critical binding agent in the mass, must be immune insofar as possible from chemical disintegration caused mainly by volumetric deformation in the formation of chemical combinations of certain elements in the cement with others, such as sodium and magnesium sulphate, commonly found in sea water.

To ensure that the concrete will be protected against physical change and destruction from freezing, (a) it must first of all be made dense and impermeable initially as previously stated, and (b) its constituents should have as low and uniform a coefficient of expansion as possible in order to avoid excessive differential temperature stresses.

Above all, however, it must be stressed that rigid and unending control of the mixing, placing and curing of the concrete on the job are of importance equal to if not even greater than that of the determination and selection of the most suitable ingredients and mixture best adapted to resist severe climatic conditions and salt water attack. Much has been written on the importance of this or that element in cements or concrete mixtures but perhaps not enough on the importance of rigid field control in the concreting operation itself.

It is therefore to be stressed that the following discussion of the experimental program of cement and concrete tests recently conducted on the Quoddy project is offered not in any sense as a final solution to the problem of evolving a concrete resistant to cold weather and salt water but merely as an indication of the effects or tendencies of certain elements or conditions on such concrete. It is offered as a complement of and not a substitute for close field control of mixing, placing and curing.

An extensive and well equipped laboratory was set up at Eastport, as a subdivision of the Engineering Division, for the work under the immediate carge of Charles E. Wuerpel, Associate Engineer, who per-

formed a splendid task in the organization of the laboratory and in the conduct of the experimental program. The laboratory was equipped with every facility for the testing of aggregates, cements and concretes.

AGGREGATES

Since stone or gravel and sand forming as they do the great bulk of any concrete are most important elements of good concrete, it was necessary to determine sources of suitable material obtainable locally if possible, in order to avoid high freight charges on the large tonnages required. Fifty-four sources of stone, gravel and sand were located and investigated. The most favorable sources of excellent appearing aggregates available at moderate haul were subjected to the usual laboratory analysis for grading, freedom from silt, specific gravity, porosity as evidenced by per cent absorption, soundness as evidenced by the magnesium sulphate test, and resistance to freezing and thawing. Sands were also subject to the standard tensile and compressive tests by comparison of mortars made of standard Ottawa Sand with others made of the sands to be tested.

An excellent sand in quantity (125,000 cu. yd.) more than adequate for the requirements of the initial project was found in an esker at Dennysville. The extent and consistency of this formation were determined by deep test pits excavated in the formation. Much of the gravel could also be obtained there, (70,000 cu. yd. incident to procuring the sand), the remainder of the coarse aggregate to be crushed stone from diabase formations at the site of the project.

Prior to finding the required amount of suitable sand, study was made of the possibility of producing a manufactured sand of fine crushed stone. Such stone dust produced a concrete of even greater compressive strength than that with Dennysville sand (which in turn gave better results than standard Ottawa) but required for approximately equal workability about ½ bag of cement more per cubic yard of concrete. Because of the lack of workability and tendency to produce a more porous though stronger mortar or concrete, it is not recommended.

As will be discussed later under the cement and concrete studies, the aggregates from these sources were found to be resistant to freezing and sulphate action; and in proper gradation and with suitable cement made an excellent concrete well suited to its conditions of exposure. To ensure absolutely the provision of suitable aggregate throughout the work, it was our plan, if operations had gone ahead, to install a screening and washing plant at Dennysville and a crushing and screening plant at Treat Island and to furnish to all contractors on

the project their aggregate requirements. On a very large construction project such as Quoddy, with a number of different contractors, such procedure is considered essential to excellent concrete.

CEMENT

It was not our plan in the limited time available, to develop a new cement but rather to test the many standard cements commercially available. These cements extend over a wide range of physical and chemical characteristics. Some 45 cements were received and analysed, many of them with almost identical characteristics. They may be classed, however, as portland cements, high early strength portland cements, alumina cement, portland-puzzolan, synthetic puzzolan, natural cement, and blended cements.

CEMENT CHARACTERISTICS

The cements were subject to the routine tests for composition, fineness, soundness, consistency, time of set, and tensile and compressive strength (Table 1). There was close relationship between the increase in mortar compressive strength at 28 days with fineness of grinding for a number of portland cements of otherwise comparable characteristics (Fig. 2).

SALT WATER FOR MIXING

Studies were made of the effect of the use of salt water for mixing purposes on the compressive strength of cement mortar. (Fig. 3). Although increasing concentrations of sea water accelerate setting as compared to fresh water, such action is generally attained at the expense of ultimate strength and durability and is therefore to be condemned. Furthermore, concrete formed by salt solutions is less resistant to freezing and thawing action. It seems almost axiomatic therefore that only fresh water should be employed for mixing concrete.

HEAT OF HYDRATION

The heat of hydration is an important characteristic of the various types of cement. The effects of important elements affecting the heat of hydration are well illustrated in Table 2.

TABLE 2—CONTRIBUTION OF EACH PER CENT OF COMPOUND COMPOSITION TO HEAT OF HYDRATION IN CALORIES PER GRAM

Element	Age 2 Days	7 Days	28 Days
C_3S	1.0	1.14	1.25
C^2S	0.0	0.21	0.42
C_3A	1.5	2.44	2.32
C_4AF	0.4	0.20	0.11
Spec. Surf.*	2.0	2.20	2.00

*For each 100 cm²/gm. over 1200 cm²/gm. specific surface multiply by the factors shown for contribution to heat of hydration. These factors apply to cements having a heat of hydration of about 100 calories per gram.

TABLE 1—OXIDE ANALYSIS AND COMPOUND COMPOSITION OF TEST CEMENTS (SEE TABLE 1A)

Serial No.	Brand	Compound Composition				Oxide Analysis							Insol. Res.	Free CaO	Sp. Grav. Cement
		C_3S	C_2S	C_3A	C_4AF	SiO_2	CaO	Fe_2O_3	Al_2O_3	MgO	SO_3	Ign. Loss			
PC- 1	C	50	22	5.6	15.2	20.7	63.1	5.0	5.3	2.2	1.8	1.6	0.3	0.8	3.16
PC- 2	B	61	16	1.0	16.7	21.7	65.8	5.5	3.9	0.9	1.5	1.0	0.2	1.0	3.18
PC- 4	A	48	29	6.5	10.3	22.9	64.9	3.4	4.6	1.6	1.8	0.9	0.3	0.7	3.15
PC- 5	B	50	28	7.5	10.4	22.6	65.0	3.4	5.1	1.3	1.6	0.7	0.7	0.4	3.16
PC- 6	B	51	26	7.4	10.4	22.6	65.7	3.4	5.0	1.5	1.7	0.8	0.2	0.5	3.17
PC- 7	C	46	26	5.8	12.2	20.8	62.9	5.0	5.4	2.4	1.8	1.9	0.3	1.1	3.15
PC- 11	B	48	29	6.6	10.4	22.8	64.8	3.4	4.7	1.5	1.6	0.8	0.3	0.4	3.15
PC- 14	A	53	24	6.1	10.8	22.3	65.3	3.6	4.6	1.5	1.7	1.1	0.3	0.8	3.15
PC- 15	B	51	27	6.4	10.4	22.8	65.2	3.4	4.6	1.3	1.6	0.7	0.2	0.3	3.18
PC- 16	A	52	24	8.2	10.4	22.0	65.2	3.4	5.3	2.6	1.3	0.9	0.2	0.5	3.16
PC- 17	C	40	33	11.7	7.6	22.2	64.4	2.5	6.0	2.0	1.7	1.3	0.2	1.3	3.09
PC- 18	D	45	26	10.2	8.4	20.8	62.6	2.8	5.6	3.1	2.4	2.0	0.3	0.9	3.08
PC- 19	D	42	30	11.4	8.4	21.4	63.4	2.8	6.1	3.1	2.1	1.3	0.2	.07	3.11
PC- 20	B	50	24	9.3	8.5	21.3	64.2	2.8	5.3	3.1	2.0	1.2	0.2	0.9	3.09
PC- 21	D	43	26	15.2	7.3	20.3	63.3	2.4	7.3	3.3	1.9	1.2	0.2	0.9	3.11
PC- 22	A	50	20	14.4	7.8	19.9	63.8	2.6	7.1	3.4	1.9	1.6	0.2	0.5	3.09
PC- 23	E	50	19	14.7	7.9	19.8	64.7	2.6	7.2	3.0	1.6	1.0	0.1	1.6	3.10
PC- 24	H	Aluminous				8.7	38.0	14.3	39.2	1.5	0.3	0.0	1.1	0.0	3.19
PC- 25	F	Portland—Puzzolan				31.2	47.4	3.3	6.2	4.0	2.0	5.4	16.2	2.8	2.93
PC- 26	E	55	13	5.8	17.7	19.0	63.6	5.9	3.9	2.7	1.9	1.0	0.2	1.5	3.17
PC- 27	B	66	8	7.2	9.7	20.2	66.1	3.2	4.8	1.4	2.4	1.7	0.3	1.3	3.10
PC- 29	B	57	12	13.8	7.5	19.0	64.4	2.5	6.8	3.4	2.5	1.2	0.2	1.3	3.09
PC- 31	E	73	1	10.0	7.0	19.6	66.6	2.3	5.2	2.6	2.4	1.6	0.2	1.0	3.07
PC- 32	I	Natural				27.1	31.7	2.3	4.6	18.7	2.4	10.4	16.1	0.1	2.89
PC- 33	G	Synthetic Puzzolana				48.1	2.3	11.2	34.9	0.6	0.9	2.4	85.2	0.2	2.54
PC- 35	B	48	29	7.9	10.4	22.5	65.5	3.4	5.2	1.4	2.0	0.7	0.2	0.6	3.15
PC- 37	D	45	28	6.5	11.1	21.6	62.6	3.7	4.8	3.6	1.8	0.7	0.2	1.0	3.16
PC- 44	J	62	10	5.7	12.6	19.9	64.3	4.2	5.8	2.0	2.3	1.5	0.2	1.1	3.08
PC- 45	J	49	27	2.3	15.6	22.2	63.5	5.1	4.2	2.0	1.7	0.9	0.2	0.4	3.08
PC-100	ABC	53	23	4.6	14.1	21.8	64.6	4.6	4.7	1.6	1.7	1.1	0.6	0.8	3.16
PC-101	ADE	50	20	14.8	7.6	21.0	63.8	2.6	6.6	3.3	1.9	1.7	0.2	1.0	3.10
PC-102	ABCI	(PC-32 21% PC-100 79%)				22.9	57.7	8.5	4.7	5.2	1.8	3.0	3.9	0.6	3.10
PC-103	ABCG	(PC-33 15% PC-100 85%)				25.7	55.3	5.6	9.1	1.5	1.6	1.3	13.3	0.7	3.06

TABLE 1A (see Table 1, opposite page)

CEMENT FINENESS AND PARTICLE SIZE DISTRIBUTION — TENSILE AND COMPRESSIVE STRENGTHS—CEMENT MORTAR

Serial No.	Fineness			Particle Size Distribution												Tensil Strength p.s.i.						Compressive Strength p.s.i.				
				Micron Sizes—% Passing																						
	200 Mesh	325 Mesh	Surf. A. cm²/gm	7.5	10	15	20	25	30	35	40	45	50	55	60	1 Da.	3 Da.	7 Da.	28 Da.	90 Da	1 Da.	3 Da.	7 Da.	28 Da	90 Da.	180 Da.
PC- 1	94.2	87.2	1790	24	29	39	47	54	60	67	73	79	82	85	87	128	272	264	449	443	45	500	2200	3915	5630	5570
PC- 2	95.4	91.5	1800	22	28	40	49	59	65	72	78	83	88	90	91	135	343	435	471	453	565	1390	2940	4590	5920	6470
PC- 4	97.6	92.8	1810	24	30	40	49	59	64	73	78	83	88	91	93	140	318	402	425	479	275	2350	2925	4275	5840	6400
PC- 5	98.6	94.2	1750	21	26	38	49	60	67	73	79	85	89	93	95	133	267	371	458	475	120	1235	1950	3525	5310	6225
PC- 6	98.8	94.4	1790	21	27	40	51	62	68	75	81	87	92	93	94	140	258	348	413	461	235	1285	2000	4400	5800	6075
PC- 7	94.8	86.9	1830	24	30	39	47	55	58	67	73	78	83	85	87	158	257	365	438	458	100	2560	2990	4375	5170	5620
PC- 11	98.6	95.7	1810	22	26	38	48	55	62	71	78	85	88	94	96	190	290	341	428	432	430	1320	2675	4325	5740	5835
PC- 14	90.0	82.9	1550	19	24	35	43	49	54	60	65	72	75	79	83	98	183	313	410	466	165	1050	2325	3405	3015	5785
PC- 15	91.0	82.4	1460	14	24	37	46	52	57	63	70	73	79	80	82	100	218	343	389	471	90	720	1390	2655	4305	4735
PC- 16	91.6	83.2	1650	21	27	36	46	50	54	62	66	71	78	81	83	107	232	395	421	441	150	875	1810	3475	4895	5475
PC- 17	94.2	86.9	1820	24	30	37	47	54	62	67	71	76	80	83	87	122	317	358	445	482	200	1260	2575	4860	5930	....
PC- 18	98.4	95.0	2710	51	60	68	76	80	88	91	95	95	95	95	95	185	373	379	484	520	525	3040	4700	6415	6895	6920
PC- 19	86.0	78.0	1670	21	25	39	46	52	55	60	66	70	73	76	78	88	283	310	447	444	140	1325	2525	4275	5515	5935
PC- 20	99.4	95.8	2020	26	33	45	57	65	72	81	87	90	94	96	96	165	328	410	505	585	140	1700	3415	6085	6925	6970
PC- 21	88.0	79.2	1380	16	20	30	38	45	52	57	61	67	71	74	79	102	277	382	433	466	65	1100	2240	3950	4800	4825
PC- 22	94.2	86.8	1670	20	26	36	47	56	63	69	73	79	81	84	87	140	310	420	411	443	175	1275	2450	4210	5400	5435
PC- 23	89.4	79.5	1420	21	28	31	40	47	53	54	64	69	74	76	80	103	298	373	413	426	150	1085	2660	4085	4465	4385
PC- 24	80.0	70.0	1390	19	22	30	36	42	46	51	54	60	63	66	70	437	432	415	415	464	7200	8170	8440	8500	7450	7710
PC- 25	86.2	85.3	2490	40	45	53	58	65	70	74	77	79	82	82	85	120	245	300	404	448	100	650	1860	3160	3985	4765
PC- 26	90.4	82.9	1310	15	21	24	33	42	49	51	63	68	77	81	83	118	282	362	430	449	125	710	1990	3600	4405	4465
PC- 27	99.0	95.7	2270	31	38	51	61	70	77	83	87	91	93	96	96	277	417	453	505	509	1435	2950	4110	5060	6410	6225
PC- 29	98.8	94.9	2030	25	35	46	56	64	72	77	85	89	93	95	95	275	338	398	515	525	690	2625	4825	6440	6650	6810
PC- 31	95.6	92.2	1950	25	32	44	54	62	69	74	80	83	88	90	92	325	408	437	484	488	1150	2565	4045	6160	6590	6625
PC- 32	90.8	86.6	2370	37	44	56	65	72	75	78	82	84	84	87	87	0	65	150	196	250	0	75	175	865	1455	1760
PC- 33	94.1	91.9	1530	14	23	40	53	63	69	76	81	84	88	90	92											
PC- 35	92.6	85.2	1715	22	27	36	45	53	59	65	70	78	80	83	85	150	255	357	384	475	475	1680	2840	4250	5140	5210
PC- 37	93.6	88.6	1980	26	31	41	50	58	64	69	76	80	85	87	89	173	317	368	453	469[1]	767	2000	3450	5467	6160	6800
PC- 44	99.6	98.2	2650	38	46	60	74	82	88	92	95	93	95	98	98	333	410	460	475	475	1460	3933	5525	6485	6750	6725[2]
PC- 45	95.0	88.2	1840	25	30	40	48	56	63	70	76	79	82	87	88	208	310	364	453	484	175	1700	2810	4310	5000	5370[2]
PC-100	95.8	90.5	1890													143	303	385	452	472	175	1425	2370	4485	5890	6150
PC-101	90.5	81.8	1490													115	263	265	419	443	75	1700	2350	4285	5265	5175
PC-102	94.7	89.7	1960													124	265	364	424	447	50	975	1960	3840	4740	5215
PC-103	95.6	90.7	1800													118	266	350	436	489	42	1215	2165	4240	5585	6940

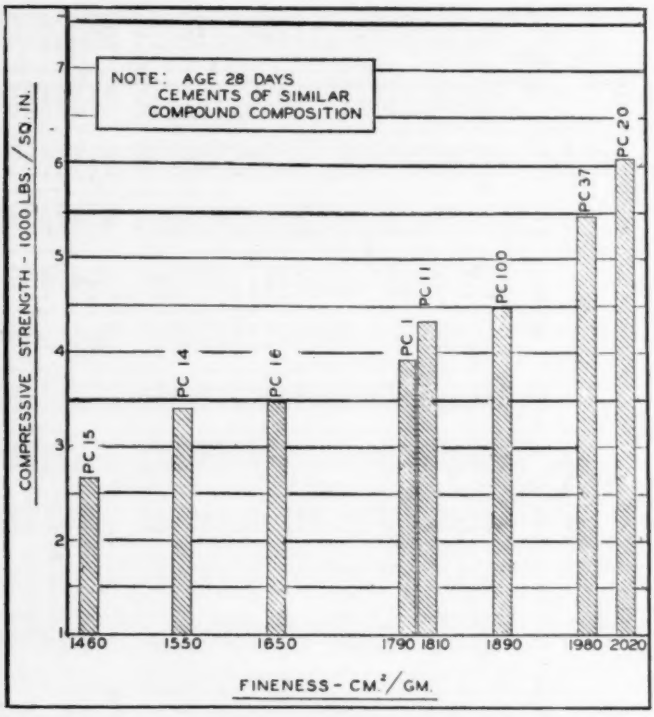


FIG. 2—EFFECT OF CEMENT FINENESS ON COMPRESSIVE STRENGTH OF MORTAR

The great effects on heat of hydration in the important initial period of setting of (a) fineness of grinding, (b) the tricalcium silicates (C^3S) and (c) the tricalcium-aluminates (C^3A) are readily apparent. These indicate the need of a balancing of these factors in the selection of the cement, as well as a close control of heat dissipation in mass and semi-mass concretes if excessive stresses, occuring during a period when the strength of the concrete has not been fully developed, with resultant cracking and incipient failure through disintegration, are to be avoided.

MAGNESIUM SULPHATE

The cement must be resistant to the chemical action of sea water. It is not possible in a short period to test the resistance of cements and concrete to sea water itself although such tests are underway with concrete columns now exposed at Eastport and will be continued). It is possible, however, to determine analogously the relative resistance of cements to sea water attack by the standard accelerated magnesium-

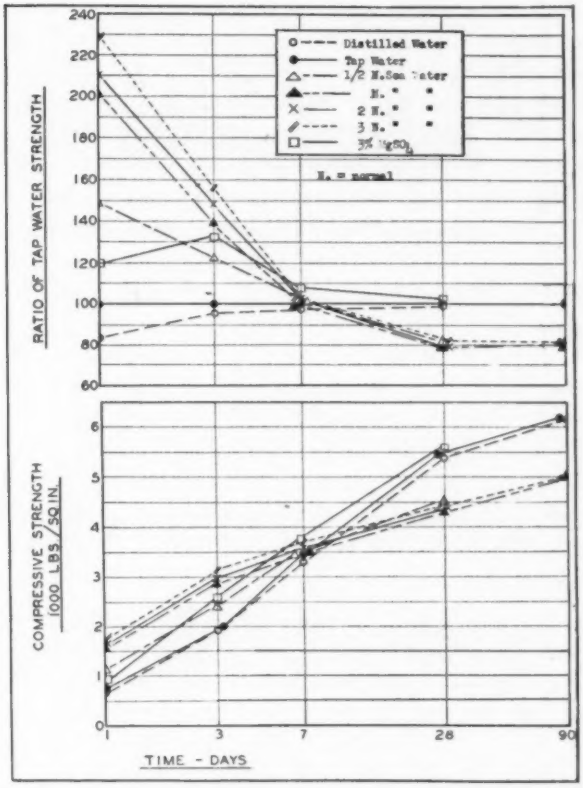


FIG. 3—EFFECT OF MIXING WATER UPON COMPRESSIVE STRENGTH OF CEMENT MORTAR (PC-37)

sulphate test by immersion of neat cement slabs in 10 per cent magnesium sulphate solutions for 280 days. Our tests (Table 3) in general confirmed the findings of other investigators that those cements high in C^3A had slight resistance to the magnesium sulphate attack and hence are not adapted to salt water use. However, the aluminous cement, which attains such high strengths in relatively short time, shows itself absolutely resistant to the magnesium sulphate action.

Our magnesium sulphate tests indicated very definitely the desirability of limiting the C^3A compound in portland cements to less than 8 per cent for proper resistance to salt water action.

BLEEDING

A concrete, including of course its cement binding paste, must be resistant to "bleeding," the expression used to designate the tendency of particles in the paste to settle so that only a film of water remains

TABLE 3—QUALITATIVE COMPARISON OF RESISTANCE OF CEMENTS TO ATTACK BY MAGNESIUM SULPHATE

Cement No.	Compounds				Fineness cm²/gm	Date Immersed	Age Days	Degree of Attack	Age Days	Degree of Attack
	C_3S	C_2S	C_3A	C_4AF						
PC- 1	50	22	5.6	15.2	1790	10-31-35	104	Slight	280	Slight
PC- 2	61	16	1.0	16.7	1800	11- 8-35	96	Slight	280	Slight
PC- 4	48	29	6.5	10.3	1810	10-25-35	110	Slight	280	Slight
PC- 5	50	28	7.5	10.4	1750	10-25-35	110	Slight	280	Slight
PC- 6	51	26	7.4	10.4	1790	10-29-35	106	Slight	280	Moderate
PC- 7	46	26	5.8	15.2	1830	10-29-35	106	Slight	280	Moderate
PC-11	48	29	6.6	10.4	1810	10-25-35	110	Slight	280	Slight
PC-14	53	24	6.1	10.8	1350	10-25-35	110	Slight	280	Slight
PC-15	51	27	6.4	10.4	1460	10-25-35	110	Slight	280	Slight
PC-16	52	24	8.2	10.4	1650	10-29-35	106	Slight	280	Slight
PC-17	40	33	11.7	7.6	1820	10-31-35	104	Moderate	280	Moderate
PC-18	45	26	10.2	8.4	2710	10-31-35	104	Moderate	280	100% Disint.
PC-19	42	30	11.4	8.4	1670	10-31-35	104	100% Disint.*	280	100% Disint.*
PC-20	50	24	9.3	8.5	2020	11-14-35	90	Slight	280	Slight
PC-21	43	26	15.2	7.3	1380	11- 8-35	96	Serious	280	100% Disint.*
PC-22	50	20	14.4	7.8	1670	10-25-35	110	100% Disint.*	280	100% Disint.*
PC-23	50	19	14.7	7.9	1420	10-25-35	110	Slight	280	Moderate
PC-24	Aluminous				1390	10-25-35	110	None	280	None
PC-25	Portland-Puzzolan				2490	10-25-35	110	Slight	280	Slight
PC-26	55	13	5.8	17.7	1310	10-25-35	110	Slight	280	Slight
PC-27	66	8	7.2	9.7	2270	10-29-35	106	Slight	280	Slight
PC-29	57	12	13.8	7.5	2030	10-31-35	104	Moderate	280	100% Disint.*
PC-31	73	1	10.0	7.0	1950	10-25-35	110	Slight	280	Slight
PC-37	45	28	6.5	11.1	1980	4-14-36	118	None	...	.....
PC-44	62	10	5.7	12.6	2650	4-14-36	118	Very Slight	...	.....
PC-45	49	27	2.3	15.6	1840	4-16-36	116	Very Slight	...	.....

*Completely disintegrated.

at the surface to bind the under side of overlying aggregate particles. As this water film evaporates, a tiny void area on the under surface of the aggregate particles remains, resulting not only in a weaker concrete but one permitting the access of water with resultant danger from freezing, etc. (Reference is made to the photomicrograph illustration, Fig. 4, of certain concrete slabs which were divided by the concrete saw employed in the laboratory and then polished.)

Tests for bleeding were made of 18 of the cements with pastes of various water-cement ratios ranging from 0.7 to 1.0 (with the apparatus shown in Fig. 5). Our tests indicated generally that high specific surface area or fineness, particularly in particles finer than the 7.5 micron size tended to reduce the bleeding effect.

Here again, however, it is desired to stress that the use of a cement of reduced bleeding characteristics, resistance to magnesium sulphate attack, etc., will not eliminate in any manner the need for close field control in the matter of low water content, excellent grading of aggregates and mix, rate of placing, and manner of placing and compaction. The cement factor is complementary to but no substitute for close field control.

TEST RESULTS ON TYPICAL CEMENTS

It is of course impossible within the limitations of this paper to list all or even a major part of the test results. The following tabulation summarizes the tests made on some of the cements which may be considered typical of the following classes:

Cement		C_3S	C_3A	Fineness
Portland	A (PC-100) (PC-37)	normal ∓ 50%	med. to low (8 to 1%)	med. high ∓ 1900 $\frac{cm^2}{g}$
	B (PC-101)	normal ∓ 50%	high (10 to 15%)	med. ∓ 1500 $\frac{cm^2}{g}$
High-Early Strength	C (PC-27) (PC-31)	normal to high (45 to 73%)	med. to high 5.7 to 13.8%)	high (1950 to 2710 $\frac{cm^2}{g}$)
Natural	(PC-32)			high 2370 $\frac{cm^2}{g}$
Aluminous	(PC-24)	(not calculable)		med.-low 1390 $\frac{cm^2}{g}$
Portland-Puzzolan	(PC-25) (PC-103)			high 2490 $\frac{cm^2}{g}$ 1800
Portland-natural blend	(PC-102)	(79% PC-100 + 21% PC-32)		1960 $\frac{cm^2}{g}$

The tabulation of test data* shows the various cement classes, the chemical analysis and compound composition, sieve analysis, fineness,

*The author's Table 4 will be of considerable interest to some readers. It is in 18 typewritten pages. Copies will be supplied to members of the Institute at the cost of reproduction by whatever process is suited to the evident demand.—EDITOR

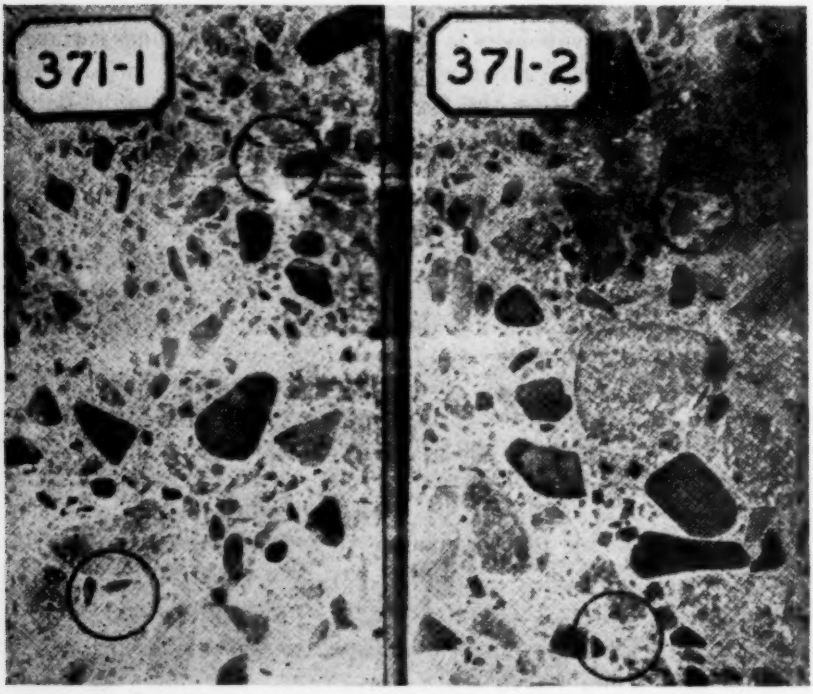


FIG. 4—CONCRETE SLABS MARKED AND READY FOR DRILLING CORES (SEE FIG. 4A)

resistance to magnesium sulphate action, volumetric change of plastic mortar during curing, under alternate wetting and drying and under alternating freezing and thawing, thermal coefficient of expansion, compressive strength under varying curing conditions, tensile strength of standard mortar, and, for concrete made with this cement, volumetric changes under different curing conditions and under alternating freezing and thawing, the coefficient of thermal expansion and compressive strength.

In general, the tabulation of test results shows that the portland cements of the A type (medium high fineness and low C^3A) attain excellent strengths in reasonably short periods and are resistant to magnesium sulphate attack and to material volume change under both alternate wetting and drying and freezing and thawing.

The portland cement Type B (medium fineness and high C^3A), though showing satisfactory strengths, could not withstand the sulphate attack nor the volume change resulting from alternate freezing and thawing conditions.

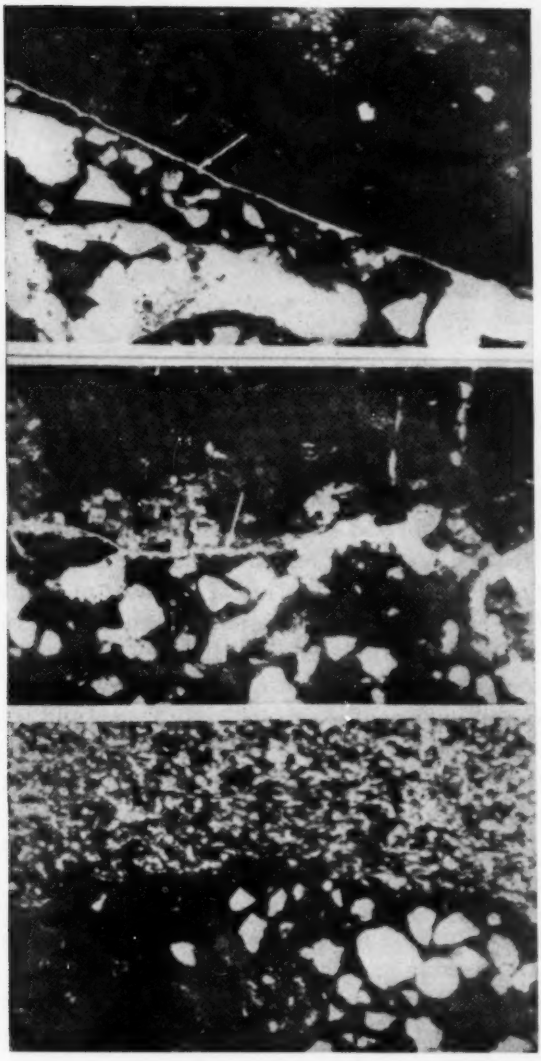


FIG. 4A—PHOTOMICROGRAPHS OF UNDER SIDES OF AGGREGATE PARTICLES SHOWING FILMS OF WATER-GAIN DUE TO BLEEDING OF CEMENTS

Top.—Normal cement; water-cement ratio, 9.90
Center.—Normal cement; water-cement ratio, 0.80
Bottom.—Non-bleeding cement; water-cement ratio, 0.90

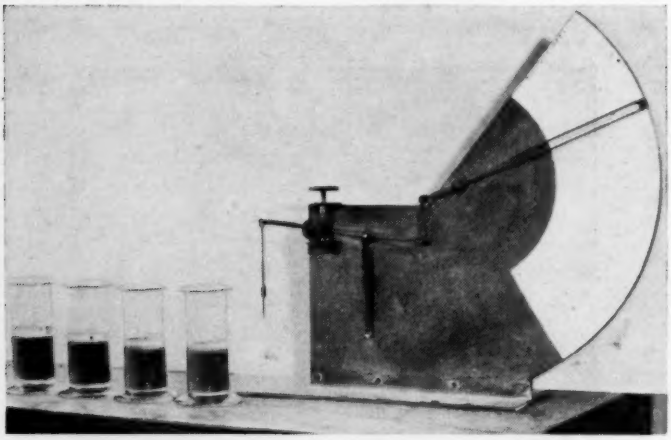


FIG. 5—BLEEDING TEST APPARATUS

The high early strength cements gave, of course, excellent strength characteristics, showed themselves but slightly affected by the magnesium sulphate attack, and resisted well the alternating wetting and drying and freezing and thawing. In mass concrete, however, there would be a problem in the dissipation of the heat of hydration.

The natural cement showed its great weakness in its slow rate of setting and in low strengths attained.

The aluminous cement gave an excellent performance throughout. It showed maximum strengths at very early stages and was absolutely resistant to sulphate attack, as well as giving an excellent performance in resistance to volumetric change from alternate wetting and drying and freezing and thawing. It showed also a low thermal coefficient of expansion. In mass concrete, there would of course be a problem in the dissipation of heat in the setting.

The portland-puzzolan cement showed reasonable strengths, gave high resistance to magnesium sulphate attack, but performed poorly under alternate freezing and thawing.

The blend of portland and natural cements showed moderate strengths, but showed poor resistance to alternating conditions of freezing and thawing.

OPTIMUM MIXTURE

The concrete studies were extended to determine for the various available aggregates the optimum mixture. This determination is most important for any major concrete structure if a dense impermeable and long-lived concrete is to be attained. It should be carried out continuously not only prior to but during the entire concreting

operation in order to make such changes as are currently indicated with change in aggregate, etc. Our studies were based on concrete to be placed by internal vibration at a rate of ± 3800 r.p.m., requiring a slump of about 1½ in. Such determinations do not of course represent the optimum for other methods of placement.

The average grading of fine aggregate employed (closely controlled) was as follows:

Sieve No.	4	8	16	28	48	100
% Passing	100	71.7	50.5	29.2	12.8	4.9

A coarse aggregate also carefully graded with maximum size of 2 in., (because of the extensive amount of closely spaced reinforcement) was also employed.

Tabulations were made of various combinations of sizes of the major coarse aggregates considered showing unit weights and voids, together with the optimum mixtures as obtained from over 150 trial batches (Tables 5 and 6).

To test density obtained—discs (¾ in. through) were sawed from the top, bottom and center of various 6 x 6 x 12-in. specimens of optimum mixes and tested for absorption, specific gravity and voids (Table 7).

When one considers that 7½ per cent of the voids occur from the excess water required for proper workability, but which fails to enter into combination with the cement, it is seen that air voids in the mix are reduced to approximately one per cent. And only by such close control can a durable concrete be obtained.

VOLUMETRIC CHANGE

Inasmuch as data on the volumetric change of concrete in setting and under extreme ranges of heat transfer including freezing, are of great importance in concrete design, an extensive series of tests relating to volumetric changes was performed on concrete beams of varying aggregates and cements. Corrollary thereto and incident to the conversion of one to the other, tests were also run to determine, in addition to the compressive strength, the modulus of elasticity and of rupture, plastic flow, thermal coefficient of expansion, diffusion constant, thermal conductivity and specific heat. Tests were run also to determine the volumetric changes caused by changed moisture conditions.

A description of these tests and discussion of their results should form the basis of a separate paper and are too extensive to be included in the scope of this article. However, some of the principal results which are of general interest are summarized.

TABLE 5—UNIT WEIGHTS AND VOIDS—COARSE AGGREGATE (COMBINED SIZES)

Material	Coarse Aggregate Ratio			Dry Weights, lbs. per cu. ft.						Per cent Voids (by Sp. Gr.)		
	No. 1	No. 2	No. 3	Loose		Rodded		Vibrated		Loose	Rodded	Vibrated
	¼" — ⅜"	⅜"— 1"	1" — 2"	% Sand	Weight	% Sand	Weight	% Sand	Weight	Same Sand Content as in Previous Three Columns		
Dennysville Gravel	33.33	66.67	..	....	97.50	....	111.00	....	117.00	41.2	33.1	29.5
"	41.67	58.33	..	....	97.75	....	111.25	....	114.25	41.1	33.0	31.2
"	20	40	40	....	108.75	....	114.00	....	118.25	34.5	31.3	28.8
"	25	35	40	....	109.50	....	116.25	....	119.50	34.0	30.0	28.0
" +Denn. Sand	20	40	40	....	.....	27.1	128.50	27.1	137.50	....	22.4	17.1
" "	25	35	40	....	.....	28.5	131.25	28.5	140.25	....	20.8	15.5
Shackford Diabase	33.33	66.67	..	....	92.50	....	103.75	....	111.00	48.1	41.9	37.8
"	41.67	58.33	..	....	92.00	....	103.50	....	110.50	48.5	42.0	38.1
"	20	40	40	....	95.50	....	109.25	....	116.00	46.5	38.8	35.0
"	25	35	40	....	96.50	....	112.00	....	116.25	46.0	37.2	34.9
" +Denn. Sand	20	40	40	25.2	114.50	25.2	131.50	28.6	138.00	34.6	25.0	21.0
" +Denn. Sand	25	35	40	26.1	113.00	23.4	126.25	27.4	133.75	35.4	28.0	23.4
Black H. Diabase	33.33	66.67	..	....	89.75	....	101.75	....	109.50	50.4	43.8	39.5
"	41.67	58.33	..	....	90.25	....	102.50	....	109.62	50.2	43.4	39.4
"	20	40	40	....	94.75	....	108.00	....	115.75	47.6	40.3	36.0
"	25	35	40	....	95.25	....	109.00	....	114.25	47.4	39.8	36.9
" +Denn. Sand	20	40	40	27.2	117.75	24.3	133.00	27.0	137.25	33.3	25.0	22.2
" +Denn. Sand	25	35	40	25.6	114.50	25.6	127.00	27.3	134.50	35.1	28.2	23.8
Devil's H. Granite	33.33	66.67	..	....	83.25	....	92.73	....	99.25	48.0	42.0	37.9
"	41.67	58.33	..	....	81.50	....	92.50	....	100.00	49.0	42.1	37.4
"	20	40	40	....	88.25	....	98.62	....	104.00	44.8	38.3	34.9
"	25	35	40	....	89.50	....	99.25	....	104.75	44.0	37.9	34.5
" +Denn. Sand	20	40	40	30.4	108.50	26.9	123.00	30.3	127.00	32.8	23.8	20.9
" +Denn. Sand	25	35	40	30.5	104.75	28.2	118.73	31.2	124.25	35.2	26.4	23.1

Dennysville Gravel = Natural Dennysville Gravel
Denn. Sand = Natural Dennysville Sand
Shackford Diabase = Crushed Shackford Head Diabase
Black H. Diabase = Crushed Black Head Diabase
Devil's H. Granite = Crushed Devil's Head Granite.

TABLE 6—OPTIMUM MIXTURES OF CONCRETE

Cement Factor Sx/c.y.	Coarse Aggregate		Max. Size	Sand No.	Cement No.	Proportions by Weight	Grad.* of C. A.	W/C gal./sx.	Sand Agg. Ratio	Slump In.	Remolding Effect Seconds to		
	Serial No.										1.0"	0.5"	0"
5.00	G-4	Dennysville Gravel	2"	S-26	PC-37	1-2.30-4.95	A	6.00	32%	1½	7	8	10
5.25	R-3	Shackford	2"	S-26	PC-37	1-2.31-4.84	B	6.00	34	1½	7	12	20
5.25	R-16	Black Head	2"	S-26	PC-37	1-2.31-4.91	B	6.00	34	1	10	18	30
5.25	R-23	Devil's Head	2"	S-26	PC-35B	1-2.31-4.34	B	6.00	34	1¼	9	19	31
6.00	G-4	Dennysville Gravel	1"	S-26	PC-37	1-2.02-3.77	H	5.50	35	3	7	9	11
6.60	R-3	Shackford	1"	S-26	PC-37	1-1.81-3.49	H	5.50	36	1¼	14	18	24

*Grading	Size No. 1 ¼"—⅜"	Size No. 2 ⅜"—1"	Size No. 3 1"—2"
A	20%	40%	40%
B	25	35	40
H	35	65	

TABLE 7—ABSORPTION, SPECIFIC GRAVITY AND VOIDS IN CONCRETE*

Coarse Aggregate	Group	App. Spec. Grav.	Abs. Sepc. Grav.	Absorption	Voids
Shackford	V-1-B	2.488	2.702	3.18%	7.92%
"	V-1-C	2.481	2.709	3.38%	8.41%
"	V-2-A	2.439	2.685	3.76%	9.16%
"	V-2-B	2.438	2.681	3.71%	9.06%
"	V-2-C	2.439	2.680	3.68%	9.00%
Dennysville	V-3-A	2.414	2.636	3.495%	8.42%
"	V-3-B	2.380	2.608	3.675%	8.74%
"	V-3-C	2.377	2.613	3.79%	9.03%
"	V-4-A	2.404	2.607	3.24%	7.78%
"	V-4-B	2.385	2.597	3.42%	8.16%
"	V-4-C	2.370	2.586	3.51%	8.35%
Average: Shackford	V-1 & V-2	2.461	2.693	3.500%	8.617%
Average: Dennysville	V-3 & V-4	2.388	2.608	3.520%	8.415%

*Optimum mixture for each aggregate—PC-37 cement.

Modulus of Elasticity

For similar concrete mixes with the portland cement type A (PC-37) and w/c = 0.8, the modulus of elasticity varied at 28 days from 3,650,000 p.s.i. for Devil's Head (granite) coarse aggregate to 5,550,000 p.s.i. with Schackford Head (diabase) aggregate with Dennysville gravel showing 4,800,000 p.s.i. These factors increased with the age of the concrete (about 25 per cent at 180 days).

Plastic Flow

Plastic flow coefficients ranged for otherwise similar optimum concrete mixes w/c = .8, under a constant load of 400 p.s.i. for 130 days from .000185 for Devil's Head granite as aggregate to .000305 for Dennysville gravel. Flow had not, however, ceased at this time when the tests had to be terminated.

Thermal Coefficient of Expansion

For similar mixes these factors ranged per degree Fahrenheit from .000004 for concrete with Schackford Head diabase to .0000045 for concrete with Devil's Head granite, just above that with Dennysville gravel at .00000445. (The coefficient for steel averages about 50 per cent higher and shows how excessive stresses may be set up between the concrete and reinforcement under wide temperature ranges.) With similar coarse aggregates (Dennysville gravel) and different cements the coefficient ranged from .00000445 (with PC-100) to .0000051 (with PC-103.) Furthermore, it was observed that those concretes with high coefficients of expansion generally showed earlier signs of failure under successive cycles of freezing and thawing.

Freezing and Thawing

Consistent with the foregoing, it was observed that the volume change, as measured by the ratio of length expansion to total length, of concrete beams under 200 and more freezing and thawing cycles varied to some degree with the same cement (PC-100) and different coarse aggregates from .00001 in. per in. to .00015 in. per in. With similar aggregate (Dennysville gravel) these coefficients ranged from .00015 with PC-100 cement to a range of .00044 to .00051 in. per in. for other cements (PC-25, 102 and 103) and to failure with one cement (PC-101). (Fig. 6).

Volume Change by Moisture Changes

With the same cement (PC-100) and different coarse aggregates the changes in length under alternate wetting and drying were generally similar whereas slightly greater changes occurred with similar coarse aggregates but different cements.

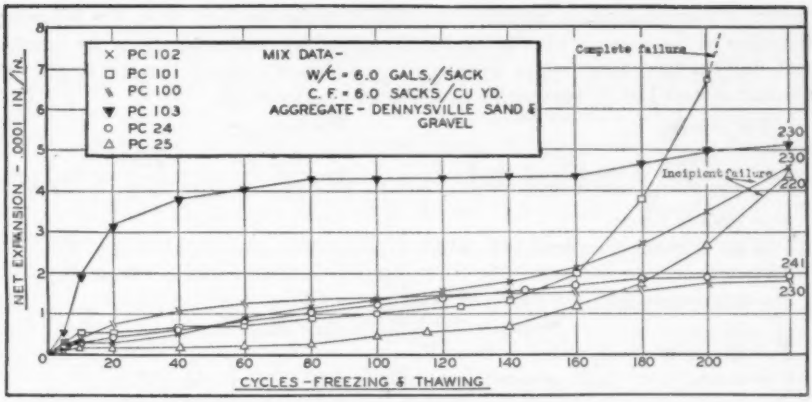


FIG. 6—FREEZING AND THAWING EFFECT OF CEMENT TYPE UPON VOLUME MOVEMENT OF 3 X 3 X 12 IN. CONCRETE BEAMS

For concrete prisms 12 x 12 x 36 in. to 48 in hermetically sealed except at one or both ends, average shrinkage ranged at 140 to 180 days from about .00027 in. per in. at ½ in. depth from the exposed end, to about .00008 in. per in. at 60 days and thereafter at 2.5 in. depth, to negligible shrinkage at 12 in. depth or greater. Comparative tests to determine the effect of steel reinforcement on the drying shrinkage of concrete were initiated but could not be brought to completion. However, these tests, extended over 80 days, indicated but little change at equal depths between reinforced and non-reinforced concrete. These tests do exemplify, however, in addition to the differences in shrinkage resulting from the use of various types of aggregates and cements, the great shrinkage which occurs during setting in the outer rim of any concrete pour, particularly if curing is neglected. Excessive tensile stresses are thereby set up during the early setting period, when the concrete is weak, in this critical outer section. In place of being the most important outer armor of protection, it thus becomes the Achilles heel or vulnerable zone for possible future destruction or disintegration of the entire mass through its tiny fissures and cracks, permitting the entry of water, salt, frost and other destructive agents.

Flexural Strength

For similar mixes flexural strength ranged as follows for corresponding coarse aggregate: For Devil's Head granite 417 p.s.i., Blackhead diabase 580 p.s.i., Dennysville gravel, 605, and Schackford Head diabase 625 p.s.i.

Thermal Flow

An extensive study of thermal flow in concrete was also undertaken by E. A. Wilder, Junior Engineer, Assistant in the Concrete Laboratory. This included a study of average temperatures to be encountered annually, to determine the number of freezing cycles together with a study of tidal cycles to determine the varying periods to which various elevations of the concrete would be subject to alternate wetting, drying and freezing. Thus with data as to air and water temperatures and convection factors and data on conductivity, specific heat, density, uncombined moisture and coefficient of surface cooling of the concrete, it is possible to determine the areas subject to alternate freezing and thawing.

Mr. Wilder evolved, in addition to other formulae on heat flow, the following formulae on the assumption that no heat is gained by conduction from the interior and that the heat liberated by freezing of uncombined moisture is equal to the net amount of heat lost by conduction in the same time interval:

For freezing:

$$t = \frac{144\,\mu\,\rho}{(28.3 - b)\,ak}\left(\frac{ax^2}{2} + kx\right)$$

For thawing:

$$t = \frac{144\,\mu\,\rho}{(b - 28.3)\,ak}\left(\frac{ax^2}{2} + kx\right)$$

t = time in hours

b = air or water temperature to which exposed (For example, air = 28° F. water = 40° F.)

a = convection factor air = 2 BTU/□ft./hr./ F.
water = 260 BTU/□ft./hr./ F.

Freezing point of sea water = 28.3° F.

k = conductivity factor concrete (example = 1.07 BTU/ft./hr./°F.)

ρ = density concrete (example 158.9 lb./cu. ft.)

μ = uncombined moisture (example 4%)

x = depth of freezing in feet

For concrete subject to tidal action, the maximum freezing under assumed constant temperature conditions occurs where the concrete frozen in t hrs. is thawed in 12.4—t hrs. (the 12.4 being the hours in a tidal cycle).

Substituting t and 12.4—t in the equations and solving show for the data assumed in this tide cycle a depth of freezing of 2.1 inch occurring at a point exposed 11.23 hrs. and covered 1.17 hrs. Formulae were also evolved to determine the rate and extent of heat flow in concrete.

Thermal Properties

The thermal properties of concrete made with different coarse aggregate were determined (Table 8).

TABLE 8—THERMAL PROPERTIES OF CONCRETE AT 70° F.

Coarse Aggregate	Conductivity "k" BTU/ft./hr./°F.	Specific Heat "C" BTU/lb./°F.	Density "ρ" BTU/lb./°F.	Coefficient of Diffusion "h²" sq. ft./hr.	Coefficient of Surface Cooling "S"
Dennysville Gravel	1.30	0.23	152.8	0.037	0.148
Devils Head Granite	1.24	0.23	149.4	0.036	0.153
Black Head Diabase Treat Island Diabase	1.06	0.23	158.9	0.029	0.161
Shackford Head Diabase	1.01	0.23	157.5	0.028	0.165

Notes: Specific heat, density, and coefficient of diffusion were determined for Class B concrete (Water-Cement Ratio = 6.0 gal. per sack. Maximum size of coarse aggregate = 2-inches) The conductivity and the coefficient of surface cooling were computed from the other thermal properties. $\left(h^2 = \frac{k}{C\rho}\right)$

CURING

In connection with the actual field placing of the concrete too much stress cannot be placed on the subject of proper curing. Irrespective of the care taken in selection of aggregates, method of mixing and placing, etc., incipient failure and limited life to the concrete may result from faulty curing.

SALT WATER CURING

Due to the expense and possible shortage of fresh water and the availability of sea water at Eastport, study was made of the use of salt water for curing purposes. No damaging effects were observed. On the contrary a comparison of 46 tests based on 28-day cylinders showed an average of 4340 p.s.i. compressive strength for cylinders cured 28 days in salt water at 70° F. as compared to 4169 p.s.i. for concrete cured in the most room 14 days at 70° F. It is most important, however, that the film of salt deposited on horizontal construction joints during curing be thoroughly cleaned off prior to concreting the next lift as otherwise a plane of weakness and future disintegration will oocur.

CURING PROCEDURE FOR MASS AND SEMI-MASS CONCRETE

For the prevention or reduction of surface cracking from rapid cooling and drying, the following procedure for mass or semi-mass concrete is warranted. Initially, the cement should not have an excessive heat of hydration incident to too high fineness or specific surface area and high C^3S and C^3A content (against these must be balanced the advantages of early strength gain, a function of these factors, and bleeding, an inverse function of fineness); secondly, the size of pour and of monoliths should be limited; thirdly, ingredients entering the concrete should be kept at a reasonably low temperature (50° F.); fourthly, to avoid excessive tensile stresses at periods when the concrete has not developed strength adequate to withstand them, forms of adequate insulating value should be kept on sufficiently long and in freezing weather such supplemental protection afforded as to assure a gradual and fairly uniform heat reduction; fifthly, maintenance of the concrete at near 50° F. for a period of about 3 days after removal of forms with a gradual reduction therefrom in winter; sixthly, to ensure against cracking from early drying of the surface concrete, the concrete should be kept saturated not only while the forms are in place but also after their removal for a total period of 14 days if possible.

For salt water concrete even greater than normal protection in the amount of concrete coverage outside of the steel reinforcement is warranted (minimum of 6 in. if possible) to ensure that the steel reinforcement is not subject to salt water attack which might occur through the incursion of salt water through tiny surface pores or checks.

TYPE OF CEMENT PROPOSED

As a result of the various tests performed at Quoddy and review of literature on the subject, it was decided to employ a portland cement

generally as specified under A. S. T. M. designation C77-32, modified as follows:

Chemical composition:

Upper limits	Loss on ignition	3.0 per cent
	Insoluble residue	0.65
	Sulphuric Anhydride (SO^3)	2.00
	Magnesia (MgO)	5.00
Ratio: Iron to alumina	$\frac{Fe^2O^3}{Al^2O^3}$ = not more than 1.56 not less than 0.50	
Silica (S^1O^2) not less than 21 per cent		
Compound composition (R. H. Bogue method)	Tricalcium silicate (C^3S)	not more than 55 per cent not less than 40 per cent
	Tricalcium aluminate (C^3A)	not more than 8 per cent
Fineness: not less than 1800 sq. cm. per gram not more than 2300 sq. cm. per gram (Wagner turbidimeter, ASTM, C115-34T)		

The cement as above specified (conforming generally to the PC-100 or PC-37 as used in the tests) is considered well suited for the semi-mass concrete exposed to the conditions obtaining on the Quoddy project. It can be developed at moderate cost at every standard portland cement mill in the country. It has no radical innovations or cure-alls. It has a good service record. It makes a concrete resistant to sulphate action, to material volumetric changes occurring from setting or moisture or heat changes, and to alternate freezing and thawing; it attains a reasonably early strength without excessive heat of hydration.

But above all, it is not expected that its use will permit the reduction by one iota of the other equally important ingredients of good durable concrete; namely, closely controlled grading, mixing, placing and curing.

The writer was chief engineer of the engineering division of the project. Charles E. Wuerpel, Associate Engineer, was in immediate charge of the concrete laboratory and tests. Lt. Col. Philip B. Fleming, Corps of Engineers, was District Engineer in local charge of the project. Brig. General George R. Spalding served as Division Engineer, North Atlantic Division, in supervisory charge. Major General E. M. Markham, Chief of Engineers, is the responsible head of this and all other river and harbor and flood control projects under the Army Engineers throughout the country.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by April 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for May-June, 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY 1937

Concrete Rehabilitation Work on the Uncompahgre Project*

By A. B. Reeves†

One of the principal features of the Uncompahgre Project of the Bureau of Reclamation in Western Colorado is the South Canal, which extends from the outlet portal of the Gunnison tunnel a distance of 11.4 miles in a southwesterly direction to the Uncompahgre River. This canal has a capacity of approximately 1,000 cubic feet per second and consists of 6.80 miles of earth channel, 4.06 miles of concrete lined channel, five tunnels having an aggregate length of 2,663 ft., and one concrete siphon 354 ft. long.

The South Canal was constructed during the years 1906 and 1907, and though not operated as an irrigation canal until 1909, waste water from the Gunnison tunnel was turned into the canal for the purpose of seasoning the banks in the spring of 1908. To raise the water in the earth sections, dams were constructed at various points, so that a water depth of from 4 to 7 ft. was maintained through several of the lined sections, Evidence of injury to the concrete lining soon became apparent and by the end of the 1910 irrigation season the lining was in a very bad condition at a number of points.

Of the 4.06 miles of concrete lined canal there are 500 feet with side slopes 2 to 1, 7,000 ft. with side slopes 1 to 1, and the balance had side slopes ½ to 1. Most of the lining was originally constructed 6 in. thick on both side slopes and bottom, though on some of the steep slopes an 8-in. thickness was used. Approximately one half of the lining was reinforced. The reinforced portions occurred where the side slopes were steepest, though a considerable amount of lining with ½ to 1 slopes had no reinforcement whatever.

*Received by the Institute Jan. 18, 1937 for presentation 33rd Annual Convention.
†Engineer, U. S. Bureau of Reclamation.

Specifications for the concrete work on the South Canal were in keeping with the standards of 30 years ago. The following quotation from a copy of the specifications under which this work was performed is typical:

Mixture—The concrete shall be composed of cement, sand, and crushed rock in the proportion of 1:3:6. The concrete is to be thoroughly mixed and handled in such a manner that it will be uniform after being placed in the forms. Any places that are porous due to the matrix separating from the rock shall be removed. The concrete shall be as wet as possible without being so wet that the matrix separates from the rock.

The aggregates were secured from the banks of the Uncompahgre River, and from tests made since, appear to have contained at least 5 per cent silt and other foreign substances. While it is believed that ordinary care was exercised in the construction of this lining, it is evident from the above specifications that the water-cement ratio was excessive and that a very permeable concrete of low grade, as compared with present day standards, was secured.

The concrete failures on this project appear to be due to the following causes:

One: The disintegrating action of alkali. This occurred in scattered spots only and was not uniform in any way. The spots appeared to be small at first and gradually increased in size. It is believed the source of the alkali was the shale underlying the lining, since upon investigation it was found that in some places decomposition had taken place next to the supporting material while the surface appeared sound and in good condition. It is also probable that some of the alkali was contributed by the drainage water from the Gunnison tunnel with which the canal was primed during the season of 1908.

Two: Settlement of the foundation material, thus permitting the weight of the slab and the water in the canal to break it down.

Three: Swelling of the shale under the lining, forcing the sides in or bulging the floor slab. The injury from this source extended over a period of not more than five years after the canal was completed.

Four: Pressure from surface water which found its way behind the side walls, in most cases causing horizontal cracks along the lining about two feet above the floor.

Five: Weathering, including frost action.

Fig. 1 and 2 show the condition of some of the lining, the failure of which can well be attributed principally to cause one or four or a combination of the two. Fig. 3 shows the result of the swelling shale on side walls and floor.

During the operation of the canal for a period of 25 years many breaks occurred in the lining, each of which was repaired more or

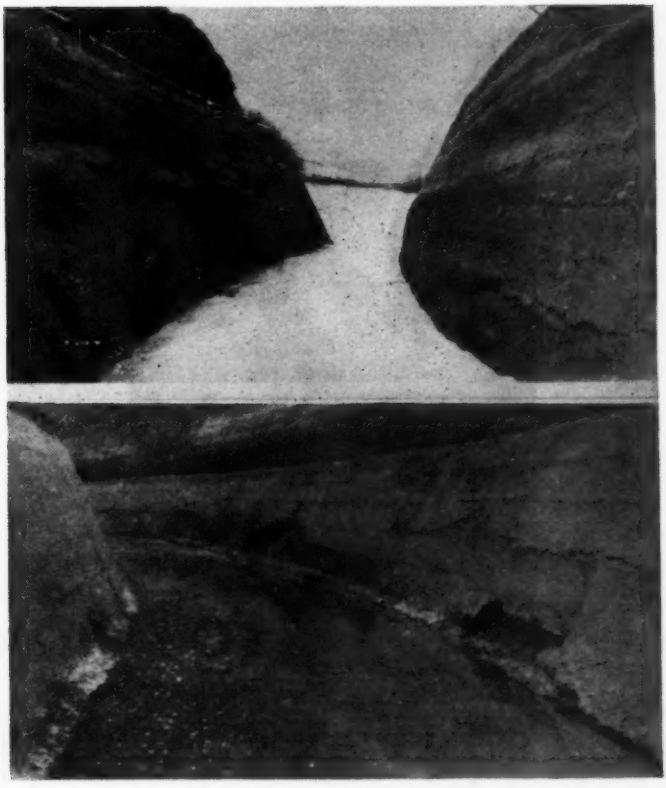


FIG. 1 AND 2—FAILURE OF CANAL LINING OF CONCRETE

less as an emergency measure. In 1934 it was decided to carry out a rehabilitation program on this project, and the repair of the concrete lining in the South Canal was one feature of this work. The portions of the canal which had given the most trouble and were consequently in the worst condition, were those in which the side walls had been constructed on ½ to 1 slopes, and contained little or no reinforcement, though there were notable exceptions to both of these cases. Much of this lining was badly broken and was unsound structurally to such an extent that any attempt at patching was considered inadequate. The method used on this type of work consisted of placing a new lining of reinforced concrete inside of the old channel, care being taken to secure as much bond or anchorage to the old concrete as possible in all cases. Fig. 4 shows a section of new lining as constructed with the keys for anchoring to the old sidewalls, and at the end of any particular section. Before the new concrete was placed care was

FIG. 3—EFFECT OF SWELLING SHALE ON SIDEWALLS AND FLOOR

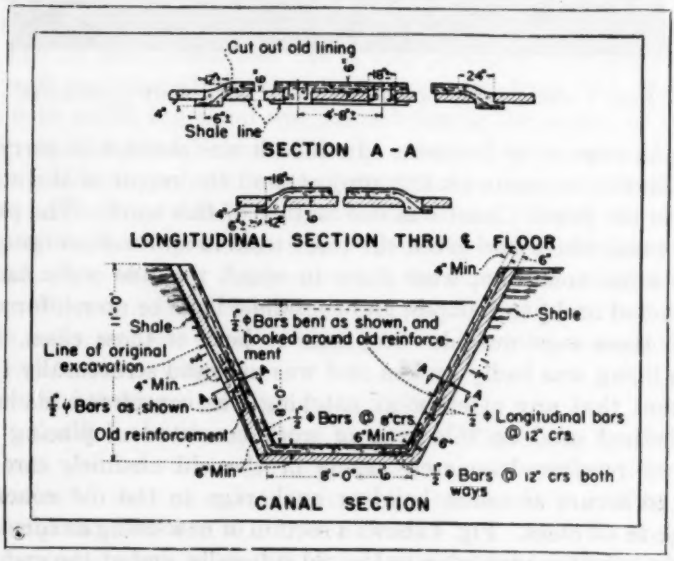


FIG. 4—SECTION OF NEW CANAL LINING

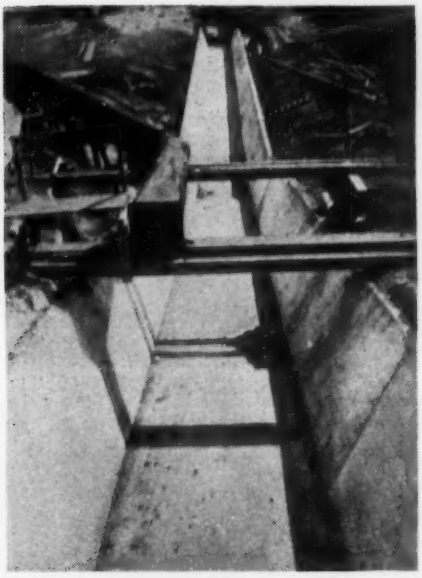


FIG. 5—SOME OF THE RELINED CANAL

exercised to see that the old surface was as clean as possible, and that no voids or hollow places existed under the original lining. In general the new reinforced section was designed to support the ordinary earth and water pressure that might be exerted back of the side walls, but no attempt was made to resist the swelling shale that had caused some of the failures when the canal was new.

Aggregates for this work were secured from the banks of the Uncompahgre River. This was, in general, the same source as for the aggregates used in the concrete in the original lining. However, in the latter case it was carefully washed and graded, the average mix by weight being 1.0 part cement, 2.1 parts sand 0 to No. 4, 1.65 parts gravel No. 4 to ¾ in., and 1.65 parts gravel ¾ to 1½ in., with a water cement ratio by weight of 0.55. The compressive strength of concrete test cylinders moist-cured for 28 days was found to average 5,500 p.s.i.

From the section shown on the drawing it will be seen that the original side slopes were followed as closely as possible; however, no actual uniform section was maintained, though as seen in Fig. 5, which shows a section of the relined canal, a good alinement was kept throughout. On account of the necessary reduction in the cross sectional

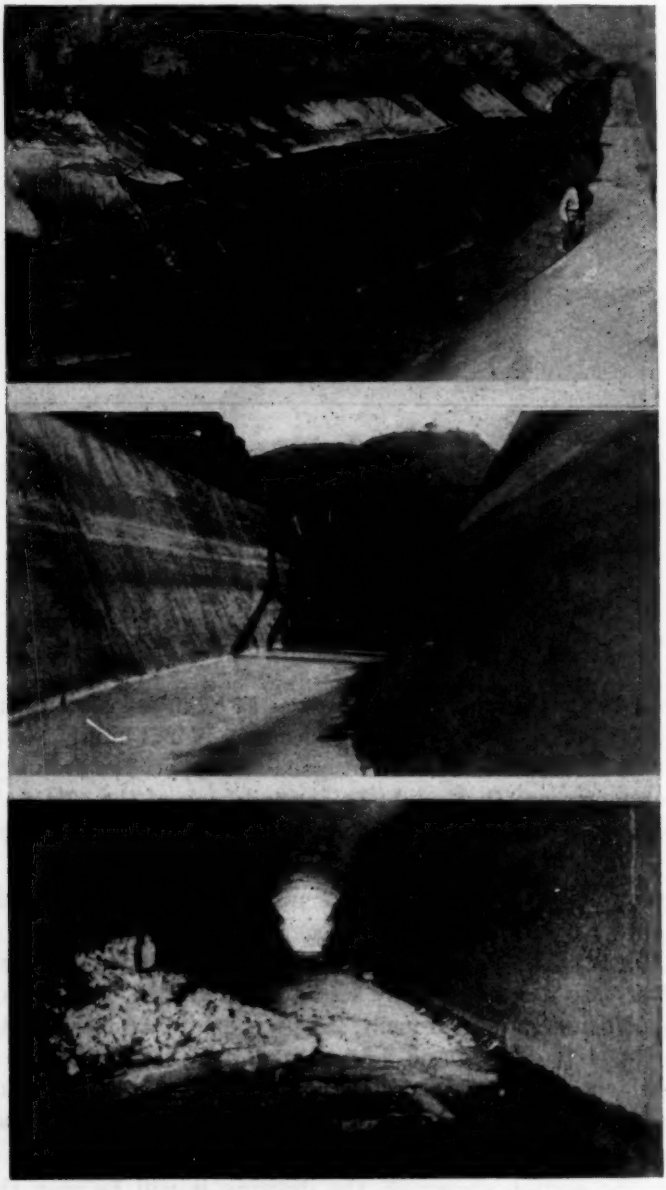


FIG. 6—BADLY CRACKED CANAL LINING

FIG. 7—LINING AS REPAIRED WITH GUNITE

FIG. 8—AN EXAMPLE OF BULGED FLOOR

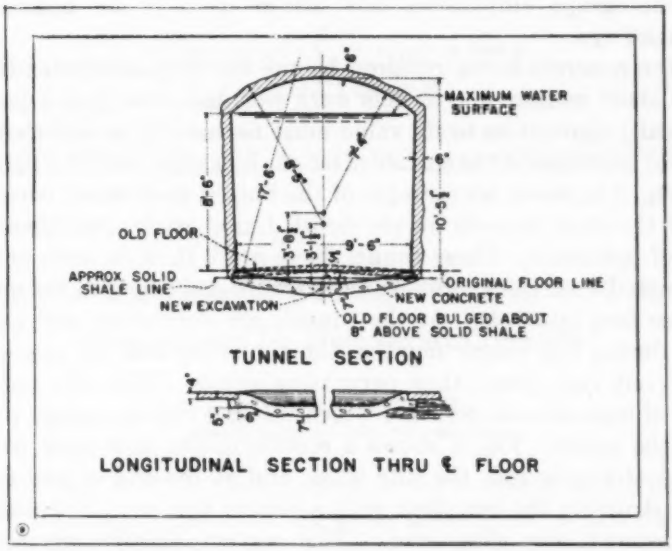


FIG. 9—SECTION OF NEW CANAL FLOOR WITH METHOD OF KEYING

area of the water prism of the canal it was important that the coefficient of roughness of the concrete be kept as low as possible in order to maintain the capacity of the canal. From observations of the flow through the relined portions of the canal during the past season it is evident that the desired results were obtained.

Fig. 6 shows a section of the canal in which the lining was badly cracked, though still in place and held together by the reinforcement. The method of repair used for lining in this condition was to cover the entire surface with gunite. The gunite layer has a thickness varying from nothing to four inches and is reinforced by bars spaced 12 in. both ways and anchored to the old lining. Before the gunite was placed the old lining was cleared of all dirt and loose concrete, and the cracks were cleared of all debris. One part by weight of cement to four parts of sand having a fineness modulus of approximately 3.00 was used; the moisture content of the sand being held between three and five per cent. The usual methods of guniting procedure were followed and by carefully regulating the percentage of water it was possible to secure a bright glazed appearance, this finish having been recommended as having a low coefficient of roughness. Fig. 7 shows a section of the lining repaired by gunite. The photograph indicates that there was a "build up" over the reinforcement bars, as shown by the slight ridges in both a longitudinal and transverse direction.

The photograph emphasizes this feature as it is not noticeable to the naked eye.

As the concrete lining repaired by the use of gunite extended over only a short section, and as this work was done less than a year ago, any final judgment as to its value must necessarily be deferred; however, the condition of the surface after one irrigation season is favorable.

In Fig. 8 is shown an example of the bulged floor which occurred in two of the short tunnels on the South Canal after approximately 25 years of operation. These tunnel floors are 9 ft. 6 in. wide and were constructed 9 in. thick. Six-in. timers were embedded in the concrete at three foot intervals. As the tunnels are very short and carry no water during the winter months, the timbering had an opportunity to dry out each year, thus permitting decay. This left but three inches of unreinforced concrete which was not strong enough to withstand the uplift. Fig. 9 shows a section of the new floor with the method of keying into the side walls, and at the end of any section. By constructing the new floor with a curved section the necessity for reinforcement was eliminated. This section slightly increases the area of the tunnel section. Inspection after one season's operation showed this repair work to be in excellent condition.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by April 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for May-June, 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY 1937

A Study of the Column Chapter of the A. C. I. Building Regulations for Reinforced Concrete*

By C. A. Willson†
MEMBER AMERICAN CONCRETE INSTITUTE

In the new A. C. I. Code it is proposed to base the safe load of a spirally reinforced column on the gross area of the section instead of the core area as heretofore. In the column investigation recently sponsored by the American Concrete Institute there is very little information to support this proposal. Out of the 573 columns tested only 14 had shells and these were round ones instead of the square shells so often used in building construction. Tests of these 14 columns are shown on page 302 of the A. C. I. Journal for January 1932. *Only four of these columns could meet the proposed requirements for amount of spiral and these four were made with the leanest mix of concrete (2000 lb. per sq. in.) and the vertical steel was less than 3% of the gross area.* Tests on four such columns do not constitute adequate experimental evidence to justify such a fundamental change in design procedure.

Two arguments have been advanced for using gross areas instead of core areas in the design of spiral columns. Both are found in a single sentence in the Tentative Final Report of Committee 105 on the Reinforced Concrete Column Investigation, in the A. C. I. Journal of February 1933, page 279, which reads: "The use of formulas based on gross areas not only produces consistency in design between tied and spirally reinforced columns but is also of decided advantage when the effect of combined axial and bending stresses is to be considered."

*Received by the Institute Dec. 21, 1936.

†Structural Engineer, State Architect's Office, Madison, Wisc. Mr. Willson, recently appointed to membership on Institute Committee 501, Standard Building Code, presents this discussion of the Institute's Code as tentatively adopted last February (Journal, Amer. Concrete Inst. Mar.-Apr. 1936 *Proceedings* Vol. 32, p. 407) in his individual capacity. He became a member of the committee about six months after the action of the convention last February.

The present practice (1928 recommendations*) of using gross areas for tied columns and core areas for spiral columns is most certainly not consistent and cannot be explained or defended easily. It upsets our sense of proportion to find that in a great many cases a tied column of a given size with a certain amount of vertical steel will carry a greater safe load than a spiral column of the same size and same amount of vertical steel when both are designed according to our present formulas. *Consistency could be re-established by basing both spiral and tied column formulas on core areas as well as on gross areas.*

With regard to the argument about bending in columns it would seem that with all of the material that has been written on the subject it ought not to be difficult for a designing engineer to determine the combined axial and bending stresses using either core or gross areas. The writer believes that the importance of bending moment in column design is somewhat exaggerated. It is probably true that an appreciable value of bending moment can be calculated for some combination of loadings on every column that has ever been built. However, in the vast majority of cases the relative importance of the actual value of the bending moment is small. On the other hand, certain critical columns in almost every building will need to be analyzed as parts of rigid frames and the bending moments will be important factors in the design of these columns. This general trend of results can be anticipated by the practicing structural engineer and appropriate methods of design can be used for the different sets of conditions. It does not seem feasible to attempt to develop a single formula which will be satisfactory for all cases.

It is believed that the importance of the small and medium size reinforced concrete column in general building construction has not been fully appreciated. Nevertheless, it is a fact that according to the 1928 recommendations a 20 by 20 in. spirally reinforced concrete column with a 17 in. round core and 3000 lb. concrete will carry a safe load of 440,000 lb. If we assume a floor panel 15 by 20 ft. carrying a total load, dead and live, of 200 lb. per sq. ft., this column will carry seven floors of it. The top story column of this seven-story building would be 12 in. square and the columns in the intermediate stories would increase to the 20 in. square column at the bottom. So it appears that the small and medium size column is used repeatedly in the field of actual building construction and this fact should be recognized in any study of design specifications.

In the past the vertical steel, spiral steel, and safe load have all been proportional to the core area of the column. In the proposed

*Superseded by the 1936 Tentative Code—EDITOR

specifications the vertical steel and the safe load are proportional to the gross area of the column and the spiral steel is proportional to the shell. The true significance of these changes in requirements is obscured by the fact that the ratio of gross area to core area varies so widely throughout the range of column sizes.

There is nothing in the 1936 regulations prohibiting the assumption of square shells when determining safe load, amount of vertical steel, and amount of spiral steel. So if we take the common case of a square column with a spirally reinforced core, it will be found difficult if not impossible to place vertical steel equal to the maximum limit of 8 per cent of the gross area given in the new recommendations.

TABLE 1

Square Outside	Round Core	Gross Area / Core Area	Per Cent of Gross Area	Equivalent Per Cent of Core Area	Steel Area Required per Inch of Perimeter
12	9	2.26	8	18.1	0.407
20	17	1.76	8	14.1	0.374
28	25	1.60	8	12.8	0.499
36	33	1.52	8	12.2	0.625

These vertical steel percentages varying from 12 to 18 per cent of the core area seem large when compared with our present maximum of 6 per cent. The steel area required per inch of perimeter indicated in the Table 1 has been found by using one ring of reinforcement in the 12 in. column and two in the larger columns with the diameter of the inner ring equal to six-tenths that of the outer. These requirements can be met in columns up to 28 by 28 in. by using 1¼ in. square rods spaced the minimun permissible distance apart. *Apparently vertical steel equal to 8 per cent of the gross area cannot be placed in larger columns in the usual manner.*

Applying the recommended formula for spiral ratio to these same columns with the useful limit stress of the spiral steel assumed at 40,000 lb. per sq. in., the results shown in Table 2 are obtained.

TABLE 2

Square Outside	Round Core	Spiral Ratio Required Concrete Strength			Spiral Ratio Provided ⅝ in. rd. at 2½ in.
		2000	3500	5000	
12	9	0.0284	0.0496	0.0709	0.0642
20	17	0.0171	0.0299	0.0428	0.0340
28	25	0.0135	0.0234	0.0337	0.0231
36	33	0.0117	0.0205	0.0292	0.0175

In a great many columns having low percentages of vertical steel the amount of spiral steel will exceed the amount of vertical steel. However, it appears that no credit is to be given for spirals less than

those indicated. This seems strange when it is realized that for the last twelve years the spiral steel has been made equal to one-fourth of the vertical steel. *Furthermore, it will be noted that even the heavy spiral indicated in the above table will not be sufficient for any size of column when a concrete strength of 5000 p.s.i. is used.*

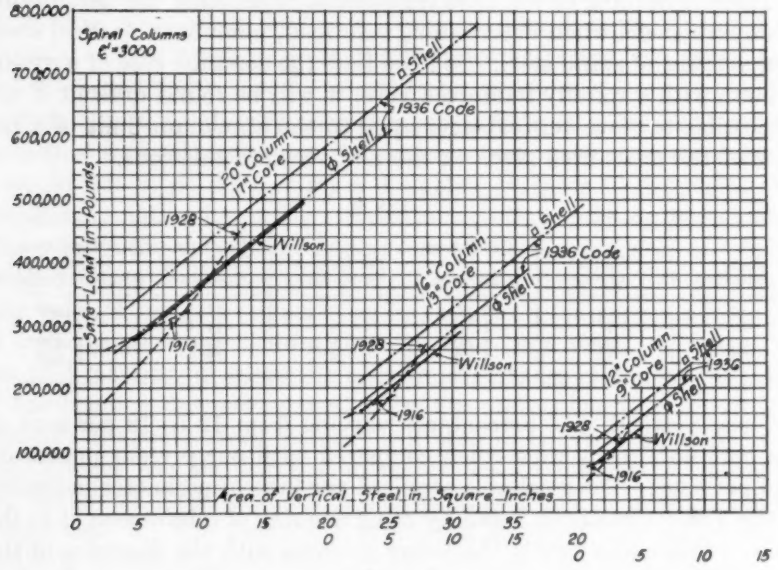


FIG. 1

If the safe load on a column is made proportional to the gross area then the protective shell of fireproofing is carrying a part of the load, —a very large part in the case of small columns. If the fireproofing shell around a reinforced concrete column core can be assumed to carry load then a similar fireproofing shell around a structural steel core ought to be permitted to carry load in the same manner. When viewed in this light the proposal to load the gross area of any column appears to be questionable.

Because of the fact that the 1936 regulations permit the vertical steel to become 8 per cent of the gross area, it will be found that the maximum safe load permitted on *small* and *medium* size columns is far in excess of that allowed by preceding specifications.

The allowable safe loads permitted by three different specifications on three sizes of columns with 3000 lb. concrete and different amounts of vertical steel are shown in Fig. 1. In the 1916 Joint Committee Report the vertical steel varied from 1 per cent to 4 per cent of the *core area*. In the 1928 A. C. I. Code the safe load and the amount of

vertical steel were kept proportional to the *core area* but the type of column formula was changed and the upper limit of vertical steel was raised to 6 per cent. According to the 1936 regulations safe load and vertical steel are proportional to *gross area* instead of core area and the upper limit of vertical steel is raised to 8 per cent. The methods of determining the amount of spiral steel vary widely in the three different specifications. However, the radical increase in safe load now being proposed results from the fact that both safe load and amount of vertical steel are made proportional to *gross area*. Comparing the new 1936 formula with the old 1928 formula, it will be noted that *the new 16 in. column will carry more than the old 20 in. column and that the new 12 in. column will carry more than the old 16 in. column.*

While neither the 1916 Joint Committee Report nor the 1928 A. C. I. Code may be regarded as perfect, yet it must be admitted that they have served as guides for the design of a great many buildings constructed during the last twenty years. It is believed that this accumulation of experience should not be disregarded and that the proposed departure from our accustomed practice should be supported by a more comprehensive knowledge than that which is now available.

The 1936 code allows the safe load on a tied column to be equal to 70 per cent of that on a spiral column. This ratio is close to that given by the 1916 specifications and it is believed that it should not exceed that value because the area of the spiral core that can be placed within a given square shell is only 78.5 per cent of the area of the tied core that can be placed in the same shell. The superior toughness of the spiral column and its ability to carry its maximum load after being materially deformed are considerations which justify the use of higher working stresses for the spiral columns than should be permitted for the tied columns.

The objections to the new column formulas can be eliminated if both formulas are kept on a core area basis as in the 1916 specification. A reasonable increase in working stresses and maximum permissible per cent of vertical steel is to be expected after twenty years of experience in the design and construction of reinforced concrete buildings.

The following formulas are suggested:

Spiral Columns

Safe Load $$\frac{P}{A_c} = 0.30 f'_c + 0.40 f'_c p_c$$

Amount of Spiral $$p' = 0.0000025 f'_c + 0.0000025 f'_c p_c$$

The 1936 regulations permit the safe load on a tied column to be equal to 70 per cent of that allowed on a spiral column. This value seems reasonable and is recommended. The ratio of vertical steel to core area may be permitted to vary from 2 per cent to 8 per cent for spiral columns and from 1 per cent to 4 per cent for tied columns. The suggested spiral column formula is shown in solid lines in Fig. 1.

If the minimum amount of vertical steel is kept to 2 per cent of the core area it will be found that high stresses in the steel induced by plastic flow of the concrete can be avoided. In addition, for many columns with square shells it will be found that 2 per cent of the core area is about equivalent to 1 per cent of the gross area.

The amount of spiral steel required by the equation suggested above is compared with that required by the 1936 building code in the following table.

TABLE 3—SPIRAL RATIO REQUIRED

Outside	Core	$f'c$ = 2000			$f'c$ = 3500			$f'c$ = 5000		
		1936	Willson		1936	Willson		1936	Willson	
		p_g = 1% or 8%	p_s = 2%	p_s = 8%	p_g = 1% or 8%	p_s = 2%	p_s = 8%	p_g = 1% or 8%	p_s = 2%	p_s = 8%
12 in. sq.	9 in. rd.	0.0284	0.0070	0.0130	0.0496	0.0108	0.0168	0.0709	0.0145	0.0205
12 in. rd.		0.0175			0.0307			0.0439		
20 in. sq.	17 in. rd.	0.0171	0.0070	0.0130	0.0299	0.0108	0.0168	0.0428	0.0145	0.0205
20 in. rd.		0.0086			0.0151			0.0216		
28 in. sq.	25 in. rd.	0.0135	0.0070	0.0130	0.0234	0.0108	0.0168	0.0337	0.0145	0.0205
28 in. rd.		0.0057			0.0100			0 .0143		
36 in. sq.	33 in. rd.	0.0117	0.0070	0.0130	0.0205	0.0108	0.0168	0.0292	0.0145	0.0205
36 in. rd.		0.0043			0.0075			0.0107		

SUMMARY

1. The superior toughness of the spiral column and its ability to carry its maximum load after being materially deformed are considerations which justify the use of higher working stresses for the spiral column than should be permitted for the tied column.

2. There are not sufficient experimental data to justify making the safe load and the vertical steel proportional to the gross area and the spiral steel proportional to the outer shell.

3. Consistency in design could be re-established by basing both spiral and tied column formulas on core areas as well as on gross areas.

4. Combined axial and bending stresses on the core area of a column can be determined without difficulty.

5. The allowable increases (1936 code) in safe load, vertical steel, and spiral steel are greatest in the columns which occur most frequently in practice,—namely, the small and medium size columns.

6. In many cases it will be found difficult and in some cases impossible to place the maximum amounts of vertical steel and spiral steel which have been allowed.

7. In the small and medium size columns a large percentage of the load is assumed to be carried by the fireproofing shell. This is quite contrary to our usual practice.

8. The objections to the new gross area formulas can be eliminated if both spiral and tied columns formulas are put back on a core area basis. The improvements in methods of design and construction which have occurred can be recognized by permitting reasonable increases in the allowable working stresses.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by April 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for May-June, 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MAY-JUNE 1937

Discussion of a paper by C. A. Willson

A Study of the Column Chapter of the A. C. I. Building Regulations for Reinforced Concrete*

F. E. RICHART†
(BY LETTER)

The author of the paper comments on the small number of columns in the A. C. I. Column Investigation which fully met all requirements of the A. C. I. Code, including protective shells. What is far more important is that the other tests of columns with shells showed very clearly the futility of using smaller spiral percentages, such as the author proposes. The tests thus furnish positive evidence as to the need for the spiral percentages of the 1936 Code. Furthermore, the whole series, and hundreds of other tests as well, have shown clearly that column strength can be analyzed into 3 distinct elements, as produced by concrete, vertical rods and spiral. Of these, the spiral contribution, while moderately reliable for short columns, is exceedingly variable and unreliable for slender columns or those subject to bending. Recent tests made at the University of Illinois on 72 columns with shells, subjected to various eccentricities of loading, show no lack of effectiveness of the shell, but very little effectiveness of the spiral. To say that our knowledge of column action is based on 4 column tests is, to say the least, incorrect and uncomplimentary to Committees 105 and 501.

No proof is given of the statement that "*consistency could be re-established by basing both spiral and tied column formulas on core areas as well as on gross areas.*" The core of a spiral column has physical significance when the application of load produces spalling of the shell, prior to bringing the spiral into play. There is no such action when a tied column fails, and there is no structural significance to the core of a

*JOURNAL Amer. Concrete Inst., Jan.-Feb. 1937; *Proceedings* Vol. 33, p. 311.
†Research Professor of Engineering Materials, University of Illinois, Urbana.

tied column. Wherein lies the "consistency"? The subject of bending is dismissed as of little consequence. If the author designs structures according to the 1936 code, he will find that bending in columns will enter all his calculations. I do not feel that any proposed column formula merits thoughtful attention unless the author has done a complete job, and has clearly thought out how bending is to be covered in his formulas. To compute bending stresses on core areas is illogical and uneconomical. Certainly spiral reinforcement in amounts less than called for in the 1936 Code adds nothing to bending resistance, yet the author would load up light building columns with spirals, where vertical reinforcement would be of real value in furnishing bending resistance. And it is in the small and medium sized column where bending troubles will be difficult to eliminate.

It is noted that the ratio of gross to core area differs widely. It does so only in very small columns where pouring difficulties, economy and considerations of bending and of slenderness would all point to the use of a tied column.

The author's criticism is aimed very largely at the small square column with spiral reinforcement, which certainly was an uneconomical column under previous codes, and still is, to a lesser degree, under the '36 Code. He ignores completely the great mass of warehouse and factory columns, which would be large circular columns, generally with 1½-in. shells, for which the area of shell is relatively small. He also ignores the provision in the Code which permits using a square shell for architectural purposes, but designing the column as a circular one. For round columns with the same vertical steel, the permissible loads proposed by the author are very close to those under the 1936 Code, though his column has much less spiral reinforcement. (See Fig. 1). The Code provides for equal factor of safety for round and square columns; the author evidently wants a greater factor for square columns, and can get it if desired by the procedure noted above.

Much space is used to show that in many square columns it is impractical to use the full 8 per cent of vertical steel permitted by the Code. The Code does not compel anyone to use this steel percentage, nor is it generally economical. However, columns have been built in practice with 12 per cent of vertical steel; bars have been used up to 2-in. square and they have been welded at splices. The committee raised the limit to 8 per cent to allow for progress, but with very definite spacing requirements which are just as important, just as restrictive, as the 8 per cent limit. The code will permit columns 4 feet in diameter, but that does not mean that it is mandatory to use them.

For freakish designs, using 5000-lb. concrete, with a minimum size of square column with spirals, the spiral requirement is unnaturally high. The obvious answer is not to use such a column. The use of a smaller amount of spiral than that required is undoubtedly a waste of material, with no chance of its acting except as a continuous tie, but if one must have spiral reinforcement and will sleep better by using it, he can employ a minimum spiral percentage and use the formula for *tied* columns in the design. It should be emphasized that the inclusion of shell area of concrete in computing allowable load on spiral columns must be paid for *dearly* in terms of spiral cost.

The author proposes as a "reductio ad absurdum" argument that the protective shell around a structural steel column be included as load carrying area. That is what is done in a composite column, where the encasing concrete is thoroughly reinforced to make the composite section act as a unit. This is done commonly in spite of the fact that perfect bond of concrete to large flat steel surfaces is unlikely, so that conditions are less favorable than in the ordinary reinforced column.

The author is worried because columns under the new code can be made smaller than under previous ones. Admittedly the factor of safety is less in the '36 formula than in the '28 code in some places, but it is greater in others.

The statement is made on page 315 that "the new 16-in. column will carry more than the old 20-in. column and that the new 12-in. column will carry more than the old 16-in. column." Even if the larger spiral percentage of the "new" column is neglected, if columns having the same vertical steel areas are compared the statement is untrue for all round columns of Fig. 1 and for all square columns except those with small areas of vertical steel. Of course, if all reinforcement is ignored, the statement is true with regard to the maximum limit of allowable load; such a statement, however, is no more significant than to compare a 12-in. silicon steel column with a 16-in. structural steel column. As a generalization, the statement is misleading.

In the author's proposed formulas for tied and spiral columns, no reference is made to supporting test data, factors of safety or underlying reasoning. The equation for required amounts of spiral also seems to be a synthetic formula to give percentages that the author considers desirable. No test data are given on the effectiveness of such spirals. No reason is given why 1 per cent of verticals is permitted in tied columns, but a minimum of 2 per cent must be used in spiral columns. It is stated that in many square columns 1 per cent of the gross area is equal to 2 per cent of the core area. This is far from true

for round columns or any square columns above 15 in. square, with the usual shell thickness.

Summing up, the author's lack of theoretical or experimental basis for his proposed formulas, the absence of data as to the factor of safety involved and the failure to give consideration to the bending resistance of columns are in marked contrast to the clearly stated principles of the report of Committee 105 on which the column formulas of the 1936 Code are based.

AUTHOR'S CLOSURE

In A. C. I. *Proceedings*, Vol. XVII, 1921, p. 150, there is a paper by F. R. McMillan—"A Study of Column Test Data." The 1928 spiral column formula was based largely upon the ideas brought out in this paper. Most of the test specimens of the recent A. C. I. Column Investigation were designed quite properly in accordance with the 1928 formula because that was the best information available at the time. Considering the radical differences between the 1936 and 1928 formulas it seems remarkable that there are even four columns which will satisfy the new requirements.

Anyone who is really interested in the subject of columns will do well to read Mr. McMillan's paper and the discussions of the 1924 Joint Committee Report published in A. C. I. *Proceedings*, Vol. XXI, 1925, page 414. Mr. McMillan in his paper makes the following statement:

"Unquestionably the most important and valuable studies of columns with both longitudinal and spiral reinforcement published in this country, are those by Prof. M. O. Withey, to be found in Bulletin No. 466, University of Wisconsin." All of those who attended the A. C. I. conventions in Chicago in 1936 and in New York in 1937 will clearly remember the positive manner in which Professor Withey objected to the provisions of the 1936 column formula.

The 1936 spiral column formula was intended for use with columns having round shells. When it is applied to columns having square shells the results are absurd in many instances. However, there is nothing in the code prohibiting the application of the formula to square columns and so the author has pointed out some of these absurdities. If a spiral column is never to be designed with a square shell or if it is never to be used for small columns, say less than 20 in., then those limitations should be stated definitely. As it stands now, the column chapter contains several jokers. Without mentioning each point in detail, the author advises the reader to view the comments of Professor Richart in this light.

The round columns with capitals and flat slab floors have their place in certain types of buildings just as square columns with rectangular beams and joist floors have their place in other types of buildings. The round columns may be cheaper than square columns but the two systems are not interchangeable. Generally speaking, round columns are not used for university, school, hospital, hotel, or office buildings, and this field is sufficiently important in the building industry to justify some consideration by those who are trying to write a code.

Professor Richart seems to feel that the subject of bending moments in columns will be handled properly for the first time with the adoption and use of the 1936 code. He seems to have overlooked momentarily the vast amount of material that has been written on the subject of rigid frame analysis. He has contributed his share to the subject himself. It is not likely that the 1936 code will change radically the methods of design which are now in vogue in the better offices.

A small column using 5000-lb. concrete is regarded as a freakish design. Anyone familiar with the problems and viewpoint of the designing engineer in an architectural office will realize that the 5000-lb. concrete is being used especially to secure a small column. If a larger column were possible a 3000-lb. concrete might be used.

It seems to the author that it will be difficult if not impossible to prove to building code officials and fire underwriters the reasons for the differences in load carrying capacity of the fire-proofing shell of a reinforced concrete column given by formula (22) and that of a structural steel column given by formula (25).

The author has seen such radically different column formulas apparently justified by test data that he feels exceedingly reluctant to attempt to justify his own formula in that manner. However, since it occupies an intermediate or compromise position between the 1928 formula and the 1936 formula, the test data will support it also as indicated in the following paragraphs.

The author has used the recommended formulas for ultimate load and safe load and the recommended values for coefficients with one exception. Professor Withey recommended to the author a value of 1.5 instead of 2.0 for k and this value has been used in all ultimate strength calculations. The yield point stress of the vertical reinforcement and the useful limit stress of the spiral reinforcement have been assumed to be 40,000 p.s.i. In the following table, factors of safety are shown for a wide variety of cases. In the author's formulas it will be noted that the amount of spiral does not vary with the size of column as it does in the other scheme.

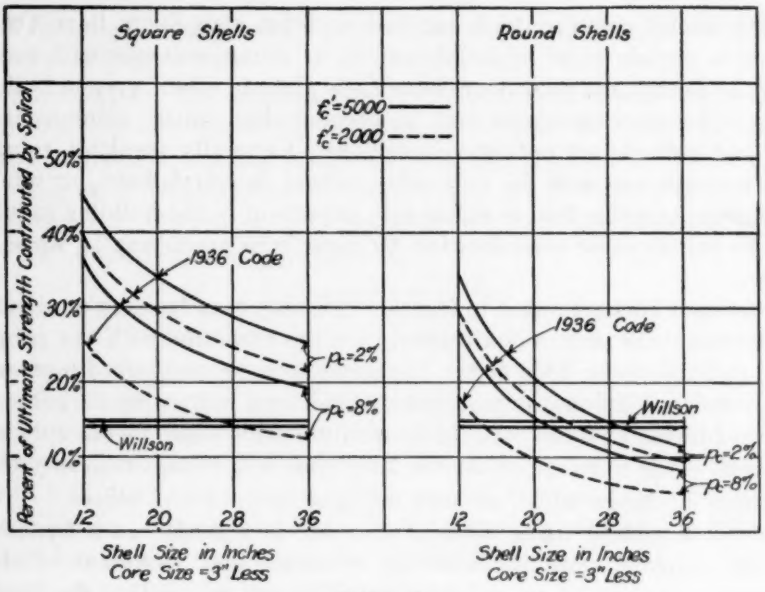


FIG. 2

FACTOR OF SAFETY BY THE 1936 CODE

Outside	Core	$f_c' = 2000$		$f_c' = 5000$	
		$p_c = 2\%$	$p_c = 8\%$	$p_c = 2\%$	$p_c = 8\%$
12 sq.	9 rd.	3.12	2.82	3.22	2.98
12 rd.	9 rd.	3.19	2.89	3.34	3.01
36 sq.	33 rd.	3.17	2.79	3.34	2.98
36 rd.	33 rd.	3.23	2.79	3.45	2.99

FACTOR OF SAFETY BY WILLSON FORMULAS

$f_c' = 2000$		$f_c' = 5000$	
$p_c = 2\%$	$p_c = 8\%$	$p_c = 2\%$	$p_c = 8\%$
3.14	2.95	3.14	3.00

There appears to be little choice between the two sets of results. Some engineers might prefer the scheme which holds the factor of safety most nearly constant.

In conclusion the author submits Fig. 2 which shows the percentage of ultimate strength of column contributed by the spiral for columns of different sizes and concrete strengths with both square and round shells. Using the provisions of the 1936 code this percent varies from 45.2 to 5.1 whereas with the author's recommendations this percentage varies from only 14.9 to 14.1 and varies with strength of concrete and amount of vertical steel and not with size of column.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY 1937

Grading and Workability*

By W. H. Glanville†
MEMBER AMERICAN CONCRETE INSTITUTE

It follows from the now well-established water-cement ratio-strength relation that the grading of the aggregate used in producing concrete is mainly of importance in so far as it affects the water-content necessary to bring the concrete to a suitable condition for placing or, conversely, for any given water-cement ratio, the amount of work involved in producing fully compacted concrete. To study the grading laws for concrete it is, therefore, essential to have some satisfactory means of measuring the condition of the concrete that determines the amount of work required. The study must therefore commence with an attempt, first to appreciate and define the physical property involved and secondly, to measure it.

The work applied in the placing of concrete may be expressed by the following equations:

applied work = work lost in shock, heat, etc. + useful work
useful work = work of compacting against internal friction + work against surface friction (i. e. concrete against boundary surfaces.)

The only work that can be regarded as a definite physical property of the mix is the work of compacting against internal friction. This leads to the definition of workability as the physical property of a concrete mix which determines the amount of useful internal work necessary to produce compact concrete.

In the measurement of workability as defined, we are confronted by two difficulties. The first is that it is impossible to measure directly the useful internal work, and the second that no matter how efficiently concrete is compacted, a certain percentage of voids will remain. The second may be overcome by stating a definite percentage, say 1

*Received by the Institute Jan. 4, 1937, for presentation 33rd Annual Convention, Feb. 23-26, 1937.
†Department of Scientific and Industrial Research, Road Research Laboratory, Harmondsworth, Middlesex, England.

per cent, as a permissible value for the voids in compact concrete but the first is virtually insuperable in the present state of knowledge.

It is, therefore, necessary to adopt a compromise and to devise a form of test which will give a measure of something which might reasonably be expected to be closely related to the amount of useful internal work necessary to produce a compact concrete. For this purpose the Compacting Factor Test has been devised. The Compacting Factor is defined as the ratio of the absolute volume of a partially compacted mix to the volume which it occupies; that is, the ratio of the absolute volume to the sum of the absolute volume and the air voids. The Compacting Factor is, therefore, a measure of the degree of compactness of the concrete in a partially compacted state.

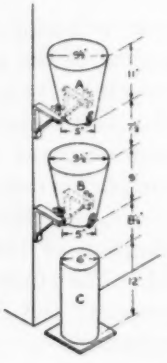


FIG. 1—COMPACTING FACTOR APPARATUS

The Compacting Factor Test measures the Compacting Factor produced by the application of a standard amount of work to concrete, and, in regarding the test as providing a measure of workability, the principal assumption is that this work is definitely related to the work required to produce full compaction of the concrete. As far as can be judged from the handling of concrete mixes the assumption is justified and it appears that reliable results are obtained which enable useful comparisons between mixes to be made.

The test must be regarded purely as a laboratory test, and not for field use. The present form of the apparatus is shown by Fig. 1. The hopper A is filled with concrete by hand from a scoop. The release of a hinged door at the bottom of A then deposits the concrete into B, a hopper with a smaller volume. Finally the concrete is released from B into C which is a 12 in. high x 6 in. internal diameter cylinder. C is filled to overflowing and the surplus is struck off by simultaneously

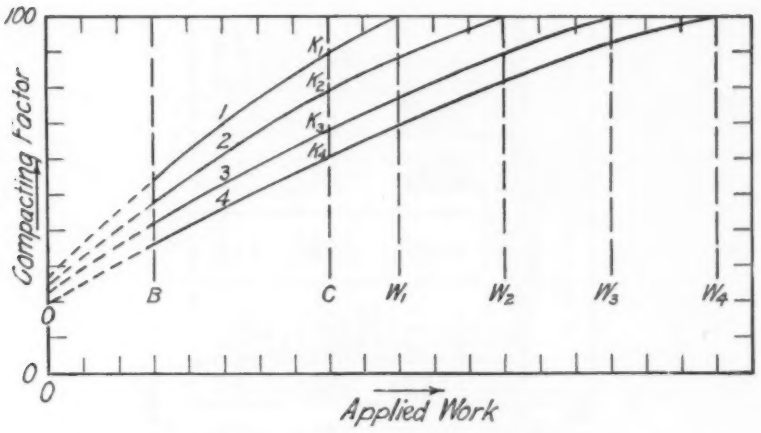


FIG. 2—THEORETICAL RELATION BETWEEN COMPACTING FACTOR AND APPLIED WORK

working two steel floats from the outside to the centre. The Compacting Factor is calculated by dividing the observed weight of concrete in the cylinder by the weight, obtained from the known specific gravities of the constituents, necessary to fill the cylinder without air voids.

The principle of the test is that the concrete is first brought to a standard state in the hopper *B*, then a standard amount of work is done on it in falling into *C* and the resultant Compacting Factor is noted. The condition of the concrete in *A* depends on the workability and the personal way of filling. This personal effect is greatly reduced when the concrete enters *B* because its Compacting Factor there is largely determined by the drop *A*-*B*. The process of the test is illustrated by Fig. 2. *OB* represents the applied working the concrete receives in being introduced into *B*. *BC* represents the work added by the fall from *B* to *C*. The measured Compacting Factors are those corresponding with the applied working *OC*. The Curves 1, 2, 3 and 4 refer to four mixes, No. 1 being the most workable, No. 4 the least workable. K_1, K_2, K_3, and K_4 are the four Compacting Factors, corresponding with *OC*, which are measured by the test (called the Standard Compacting Factor Test).

The Compacting Factor Test has been used in a series of investigations to compare aggregates of different gradings.

Various authorities have from time to time, as a result of research and experience but without any proper basis for the measurement of workability, published sets of grading curves stated to be

TABLE 1A—RIVER AGGREGATES

Workability Required	Approx. Compacting Factor	Grading	1:3 Mix				1:4½ Mix				1:6 Mix				1:7½ Mix			
			Water/Cement Ratio	Estimated Strength p.s.i.	p**	Remarks	Water/Cement Ratio	Estimated Strength p.s.i.	p**	Remarks	Water/Cement Ratio	Estimated Strength p.s.i.	p**	Remarks	Water/Cement Ratio	Estimated Strength p.s.i.	p**	Remarks
Low	.85	1 Coarse	.355	6150	108	Suitable	.43	5000	118	Suitable	.54	3600	129	Suitable	.65	2650	132	Suitable
		2	.355	6150	108		.43	5000	118		.535	3650	130		.65	2650	132	
		3	.36	6100	107		.445	4800	113		.55	3500	125		.69	2350	117	
		4 Fine	.365	5700	100		.485	4250	100	H.W.C.*	.625	2800	100	H.W.C.*	.75	2000	100	H.W.C.*
Medium	.92	1 Coarse	.375	5800	108	Suitable	.495	4150	111	Suitable	....	....	...	(Segregates	....	....	...	Segregates
		2	.375	5800	106		.475	4400	117		.625	2800	117	Suitable	...	....	...	
		3	.38	5750	105		.48	4300	115		.615	2900	121		.80	1850	103	Suitable
		4 Fine	.40	5450	100		.525	3750	100	H.W.C.*	.68	2400	100	H.W.C.*	.835	1800	100	
High	.95	1 Coarse	.395	5500	108	Suitable	....	....	...	(Segregates	....	....	...	Segregates	....	....	...	Segregates
		2	.385	5700	112		.52	3850	110	Suitable	....	....	...		....	....	...	
		3	.39	5600	110		.515	3900	111		.67	2450	111	Suitable	....	....	...	
		4 Fine	.42	5100	100		.55	3500	100		.71	2200	100		.88	1750	100	Suitable

*High Water Content.
**In column "p" the strengths are reduced to percentages of the lowest in each group with fine grading (No. 4)
The figures for strength are based on a typical strength water/cement ratio curve giving 3000 p.s.i. at w/c = 0.60
Mix ratios are by weight.

TABLE 1B—CRUSHED GRANITE

Workability Required	Approx. Compacting Factor	Grading	1:3 Mix				1:4½ Mix				1:6 Mix				1:7½ Mix			
			Water/ Cement Ratio	Estimated Strength p.s.i.	p**	Remarks	Water / Cement Ratio	Estimated Strength p.s.i.	p**	Remarks	Water / Cement Ratio	Estimated Strength p.s.i.	p**	Remarks	Water/ Cement Ratio	Estimated Strength p.s.i.	p**	Remarks
Low	.85	1 Coarse	.365	6000	116	Suitable	.465	4550	125	Suitable	.575	3250	144	Suitabe	.71	2200	133	Suitable
		2	.37	5850	113		.47	4500	125		.605	2950	131		.745	2000	121	
		3	.385	5700	111		.51	4000	111		.655	2550	113		.805	1800	100	
		4 Fine	.415	5150	100	H.W.C.*	.54	3600	100	H.W.C.*	.70	2250	100	H.W.C.*	.865	1650	100	H.W.C.*
Medium	.92	1 Coarse	.395	5500	113		.53	3700	121	Suitable	....	....	...	Segregates	....	....	...	Segregates
		2	.395	5500	113	Suitable	.53	2700	121		.685	2350	124	Suitable	....	....	...	
		3	.41	5200	108		.555	3500	115		.735	2050	108		.905	1550	103	Suitable
		4 Fine	.44	4850	100		.595	3050	100	H.W.C.*	.77	1900	100	H.W.C.*	.975	1500	100	
High	.95	1 Coarse	.41	5200	109		....	....	...	Segregates	....	....	...	Segregates	....	....	...	Segregates
		2	.41	5200	109	Suitable	....	....	...		....	....	...		....	....	...	
		3	.42	5100	107		.58	3200	114	Suitable	.785	1850	106	Suitable	....	....	...	
		4 Fine	.45	4750	100		.625	2800	100		.815	1750	100		1.03	1400	100	Suitable

*High Water Content.
**In column "p" the strengths are reduced to percentages of the lowest in each group with fine grading (No. 4).
The figures for strength are based on a typical strength water/cement ratio curve giving 3000 p.s.i. at w/c = 0.60.
Mix ratios are by weight.

TABLE 2—SUITABILITY OF GRADINGS FOR DIFFERENT MIXES

Zone (See Fig. 3)	Degree of Workability Required					
	Low		Medium		High	
	River Aggregate	Crushed Granite	River Aggregate	Crushed Granite	River Aggregate	Crushed Granite
A (Between gradings 1 and 2). Coarse	Suitable for all mixes.		Suitable for 1:4½ mixes and richer. Upper limit (Curve 2) suitable for 1:6 mixes.		Suitable for 1:3 mixes only.	
B (Between gradings 2 and 3). Medium	Suitable for all mixes.		Suitable for 1:6 mixes or richer. Upper limit (Curve 3) suitable for 1:7½ mixes		Suitable for 1:4½ mixes or richer.	Suitable for 1:3 mixes only. Upper limit (Curve 3) suitable for 1:4½ mixes.
	Upper limit (Curve 3) requires high water content with 1:6 mixes.	Upper limit (Curve 3) requires high water content with 1:6 and 1:7½ mixes.		Upper limit (Curve 3) requires high water content with 1:6 mixes.		
C (Between gradings 3 and 4). Fine.	Suitable for all mixes.		Suitable for all mixes.		Suitable for 1:6 mixes and richer. Upper limit (Curve 4) suitable for 1:7½ mixes.	
	Requires high water content with 1:7½ mixes. Upper limit (Curve 4) requires high water content with 1:4½ and 1:6 mixes.	Requires high water content with 1:6 and 1:7½ mixes. Upper limit (Curve 4) requires high water content with 1:3 and 1:4½ mixes.	Upper limit (Curve 4) requires high water content with 1:4½ and 1:6 mixes.	Requires high water content with 1:6 mixes. Upper limit (Curve 4) requires high water content with 1:4½ mixes.		

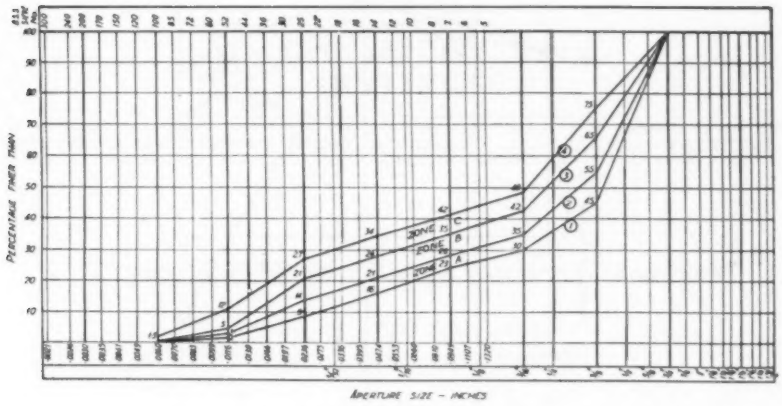


FIG. 3—GRADING OF AGGREGATES

suitable for particular purposes. These curves show considerable variations and are far from being in agreement. After careful consideration of these curves and of the work already done at the Building Research Station, a further set of gradings (given in Fig. 3) was prepared. It seems probable that these gradings would form a useful means of classifying the various mixes for different purposes.

Using these grading curves and mixes ranging from 1:3 to 1:7½, a complete exploration was made over a range of water-cement ratios sufficient to cover a change in workability from an "earth damp" mix to the point where segregation became troublesome. These tests have led to a number of very interesting and important results which are summarized in Tables 1 and 2.

Tables 1a and 1b show the water-cement ratios (and their corresponding strengths) required to produce three degrees of workability which are described as "low, medium, and high." In addition, the strengths have been expressed as a percentage of the lowest in each group.

For a 1:3 mix it is clear that the grading of the aggregate is of very small importance. The importance of grading increases progressively both with a reduction in cement content and with the requirement of a higher degree of workability. For the leanest mix, 1:7½, with a high degree of workability only the finest grading is suitable.

Table 2 shows in tabular form the mixes and degrees of workability for which the particular gradings are suitable.

It will be seen that with crushed granite aggregates, as might be expected, higher water contents are required than for river sand and

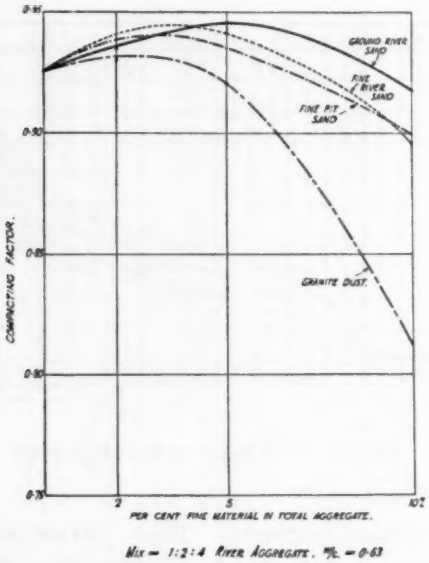

GROUND RIVER SAND
FINE RIVER SAND
FINE PIT SAND
GRANITE DUST
COMPACTING FACTOR.
PER CENT FINE MATERIAL IN TOTAL AGGREGATE.
Mix — 1:2:4 River Aggregate. W/C — 0·63

FIG. 4—EFFECT OF THE PRESENCE OF FINE MATERIAL ON THE WORKABILITY OF CONCRETE

gravel and that in general for equal workability finer gradings or greater cement contents are necessary.

An investigation has also been made of the effects of the presence in the aggregate of fine material of different types. The tests were made using a concrete mix of proportions 1:2:4 by weight made with river sand and gravel aggregates and with a water-cement ratio of 0.63 by weight. The sand consisted of equal parts of the sizes between B. S. I. sieves 100-52, 52-25, 25-14, 14-7, 7-4, and the gravel of one part of $\frac{3}{16}$ to $\frac{3}{8}$ in. and two parts of $\frac{3}{8}$ to $\frac{3}{4}$ in. The Compacting Factors are shown in Fig. 4. They indicate that up to 5 per cent of fine material there is in general some improvement, above 5 per cent there is a falling off in workability, particularly marked in the case of granite dust. These tests are of course not conclusive as the results must depend on the grading of the mix in which it is used.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by April 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for May-June, 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY 1937

Drying Shrinkage of Large Concrete Members*

By Roy W. Carlson†
MEMBER AMERICAN CONCRETE INSTITUTE

An investigation to determine the volume change in concrete members of varying size was begun at the Massachusetts Institute of Technology in the fall of 1934. The more specific aim of the investigation was to determine the properties of concrete necessary for the prediction of volume changes due both to drying and to temperature change for structural members. A portion of this investigation has been completed.

As a first step, the present paper describes the application of diffusion principles to the problem of computing approximate drying shrinkage in members of various sizes. The flow and distribution of moisture is computed by the same method as has been described by Glover[1] for computing the flow of heat. The suitability of the method for predicting approximate states of drying is indicated by test data on specimens that have been under observation for two years. Some of the specimens are large enough that in two years time no shrinkage has developed in the interior portions.

THEORETICAL CONSIDERATIONS

The diffusion of moisture from the interior of concrete toward a drying surface can be likened to the diffusion of heat from the interior of a warm solid to the cooler surface. The solutions of heat equations, that have been adequately described by Glover[1] and others, serve adequately for moisture problems if moisture concentration is considered to be analagous to temperature and the diffusivity of moisture is analagous to diffusivity of heat. The amount of moisture that will evaporate from the concrete under given temperature and humidity

*Received by the Institute Jan. 12, 1937.
†Associate Professor of Civil Engineering, Mass. Institute of Technology, Cambridge, Mass.
[1]Glover, "Flow of Heat in Dams," Journal Amer. Concrete Inst., Nov.-Dec. 1934; *Proceedings* Vol. 31, p. 113.

conditions is defined "evaporable" moisture. The concentration of evaporable moisture then replaces temperature in the equations. Similarly, a moisture-diffusion constant replaces the heat-diffusion constant.[2] Such a constant, which enters all equations, defines the speed with which a porous solid like concrete can approach an equilibrium state of drying under given conditions.

The moisture-diffusion constant is the amount of moisture, in terms of total evaporable water in a cubic foot, that diffuses through a one-foot cube in a day when there is a unit change in moisture content through the cube. Suppose that moisture is supplied to one face of the cube to keep it from losing any evaporable moisture, and moisture is evaporated from the opposite face as fast as it appears, the sides being sealed. If the amount of moisture that is evaporated in one day under such conditions is equal to the total amount of evaporable water in a corresponding, undried cube, the diffusion constant is unity. It will be shown later that the rate of diffusion through average concrete is roughly one-tenthousandth this great and the diffusion constant is therefore roughly 0.0001 ft.^{2} per day. The dimensions, feet squared per day, are the same as for heat flow and are necessary because, in general, it is desired to consider shapes other than one-foot cubes. It is instructive to note that under the above definition for diffusion constant, a concrete having twice as much evaporable moisture as another concrete and twice as rapid a rate of moisture loss, has the same diffusion constant.

It is not the intention here to present the mathematics involved in diffusion problems, but rather to give the results from mathematical studies to compare with test results. The mathematics have already been well treated in the paper of Newman, referred to above. Curves will be presented that permit solution of any common problem involving slabs or rectangular prisms without the further use of equations.

In reality, the moisture-diffusion "constant" of concrete is a variable and is only treated as a constant for convenience. The limitations should be borne in mind. In the first place, a fully saturated concrete will give up about half of its evaporable water readily but will give up the remainder with increasing difficulty. This causes a progressive reduction in diffusion constant that must be considered if final stages of drying are involved. Also, the moisture-diffusion constant varies with age of moist curing of the concrete, because not only does the amount of evaporable water vary but the pore structure varies also. These variables affect the diffusion oppositely but are not fully com-

[2]Newman, "The Drying of Porous Solids," Interim Publication Am. Inst. of Chem. En., Aug. 1, 1931.

pensating. For these reasons and for others not mentioned, great accuracy should not be expected in computing the distribution of moisture in a concrete member subject to drying.

The moisture in the interior of drying concrete drifts toward the surface as a vapor. The concentration or humidity of the moisture vapor increases with depth below a concrete surface. Liquid water remains in only the smaller pores near the surface but fills larger pores at greater depths and near a given point tends to fill all pores too small to permit evaporation at the existing humidity near that point. The liquid water in the smallest pores will never leave at ordinary humidities, but only under high vacuum or high temperature, and then very slowly. Thus the picture of drying concrete is that of a porous solid with at least the larger pores free of liquid water but full of moist air to as great a depth as the drying has penetrated. The dampness of the air in the pores increases with depth until, for an initially saturated concrete, it reaches 100 per cent relative humidity at the greatest depth of drying. It is the tendency for the dampness to equalize that causes the slow drift of moisture toward the dry surface. At ordinary temperatures this is an exceedingly slow process because, while the water is stored in concentrated liquid form in the small pores, it drifts toward the surface as a rare vapor.

EVALUATION OF MOISTURE DIFFUSION CONSTANT

In Fig. 1 are presented a group of curves showing the rate of drying of 1½ by 1½ by 12-in. bars of neat cement and concrete. The per cent of evaporable water, based on the total determined from separate tests, is plotted against days of drying. Note that some of the specimens lost about 50 per cent of their evaporable water in one day but were very slow in giving up the last few per cent. The specimens represented cover a wide range of porosity, from dense, neat cement of water cement ratio .30, to concrete containing cement labeled "blend," in which 20 per cent of the so-called "standard" cement was replaced by an equal weight of silica gel.

In order to convert the rates of drying shown in Fig. 1 to diffusion constants, several theoretical curves have been added. These represent the computed rates of drying of 1½-in. square prisms, assuming uniform diffusion constants of .000010, .000050, and .000200 ft.^{2} per day. It is seen that the shapes of the theoretical and observed curves are different, as would be expected if the diffusion constant diminishes as drying progresses. While it would be possible to determine and use a variable diffusion constant with fair accuracy, it is believed that for practical purposes, average values can be chosen that are reason-

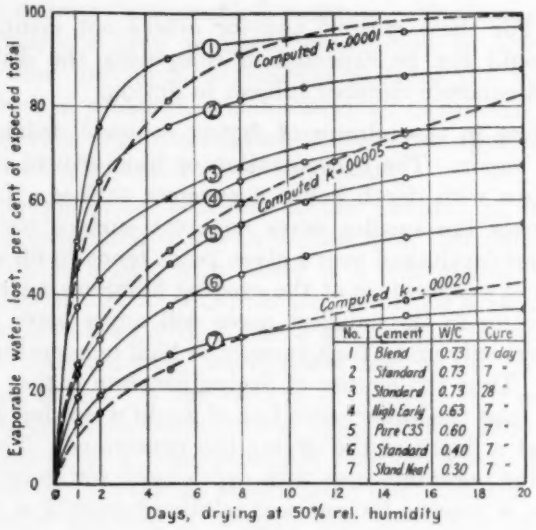

Evaporable water lost, per cent of expected total
Days, drying at 50% rel. humidity
Computed k=.00001
Computed k=.00005
Computed k=.00020
No. Cement W/C Cure
1 Blend 0.73 7 day
2 Standard 0.73 7 "
3 Standard 0.73 28 "
4 High Early 0.63 7 "
5 Pure C3S 0.60 7 "
6 Standard 0.40 7 "
7 Stand Neat 0.30 7 "

FIG. 1—RATES OF DRYING OF SMALL SPECIMENS OF CONCRETE AND NEAT CEMENT

ably reliable over the range involved. Referring to Fig. 1, it may be seen that average concretes have a diffusion constant of more than .00020 ft.^{2} per day for the first 50 per cent of drying, but that for the later stages of drying the constant may be less than .00005 ft. per day. Thus, for approximate applications, and for drying less than about 90 per cent of the ultimate, a value of .00010 ft.^{2} per day appears to suit the average concrete.

RELATION BETWEEN DRYING AND SHRINKAGE

In general, shrinkage is not proportional to the amount of water lost, since the early evaporation of free water causes little shrinkage. However, it was found that for the average concrete containing only the original mixing water and cured for a month or more, the shrinkage was approximately proportional to the amount of water lost. This is in agreement with the results reported by Menzel on standard-cured concrete containing calcareous gravel aggregate.[3] Therefore, when both moisture loss and drying shrinkage are stated in terms of per cent of expected total, they should be nearly identical.

TEST RESULTS ON LARGER SPECIMENS

Shrinkage due to drying at 50 per cent relative humidity and 80° Fahrenheit, was measured on specimens of three sizes as follows:

[3]Menzel, "Strength and Volume Change of Steam-Cured Portland Cement Mortar and Concrete," JOURNAL Amer. Concrete Inst., Nov.-Dec. 1934; *Proceedings* Vol. 31, p. 125.

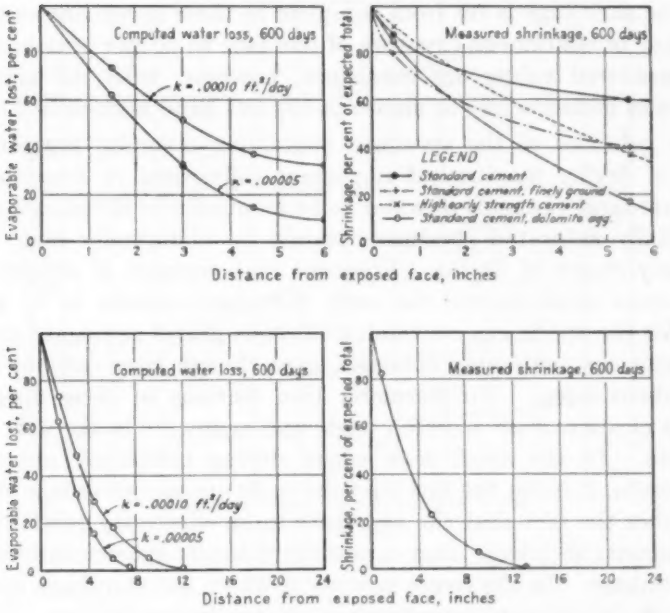

Computed water loss, 600 days
Evaporable water lost, per cent
k = .00010 ft.²/day
k = .00005
Measured shrinkage, 600 days
Shrinkage, per cent of expected total
LEGEND
Standard cement
Standard cement, finely ground
High early strength cement
Standard cement, dolomite agg.
Distance from exposed face, inches
Computed water loss, 600 days
Evaporable water lost, per cent
k = .00010 ft.²/day
k = .00005
Measured shrinkage, 600 days
Shrinkage, per cent of expected total
Distance from exposed face, inches

FIG. 2—COMPARISON BETWEEN COMPUTED DISTRIBUTION OF DRYING AND MEASURED DISTRIBUTION OF SHRINKAGE IN 6-IN. CUBES OF CONCRETE DRYING FROM ONE FACE ONLY

FIG. 3—COMPUTED DISTRIBUTION OF DRYING AND MEASURED DISTRIBUTION OF SHRINKAGE IN PRISMS 2 FEET LONG DRYING FROM ONE END

a. Small bars, 1½ by 1½ by 12-in., to determine the ultimate shrinkage to be expected.

b. Six-inch cubes, cast in thin copper forms with one face open; these specimens represent 12-in. slabs drying from both faces.

c. Prisms two feet long, cast in thin copper forms with one end open; these specimens represent 48-in. slabs drying from both faces.

Representative tests on the small bars that were companions to the cubes for which test results are to be shown, give the following results:

Identification	2-Year Shrinkage
Standard Cement	.00052
Finely Ground Cement	.00054
High Early Strength Cement	.00055
Standard Cement, Dolomite Aggregate	.00035

Assuming these values to define the ultimate shrinkage to be expected in the larger specimens, relative shrinkages as measured on four of the cubes are shown in the right hand portion of Fig. 2. Note

that the shrinkage is far from complete in these specimens even after 600 days of continuous exposure of one face to 50 per cent humidity. The measured values are consistent, however, with the computed degrees of moisture loss as shown in the left hand portion of Fig. 2.

The behavior of the specimen containing dolomite suggests that rates of drying may reveal important differences in concretes. If there are large channels of water to be emptied, the diffusion constant of initially saturated specimens should be abnormally high during the early stages of drying. Frequent measurement of weight losses in a forced draft showed the early diffusion constant to be greatly more for the specimens containing siliceous gravel aggregate than for the specimens containing dolomite, even though later rates of drying were about equal. Furthermore, thin sections of these specimens showed closer contact between paste and aggregate in the case of the dolomite. In the small bars where drying ultimately approaches equilibrium, it is not the fact that the moisture cannot escape readily, but rather the fact that the aggregate more effectively reinforces the paste against shrinkage, that causes the dolomite specimens to exhibit low shrinkage. In the larger specimens where the shrinkage does not approach the ultimate for several years, the lower diffusion constant for the earlier stages of drying is effective in retarding the drying from the interior.

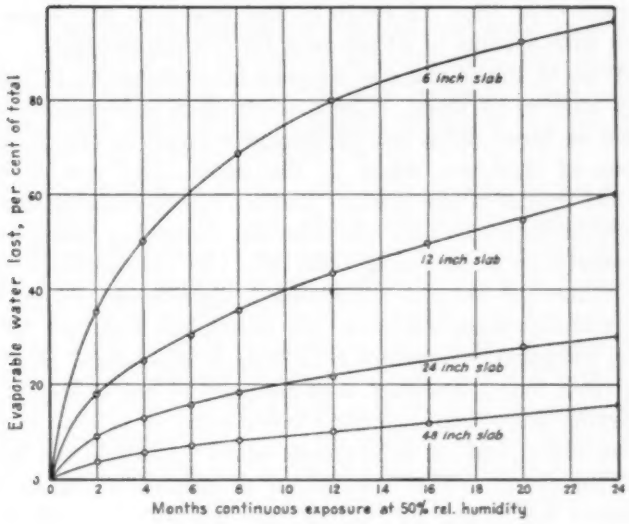
In Fig. 3, the results are shown for a prism two feet long, drying from one end only. In the left hand diagram, computed amounts of moisture loss at 600 days are plotted for various depths from the drying surface, assuming two different diffusion constants. In the right hand diagram, measured shrinkage is shown for the same period of continuous exposure at 50 per cent humidity. It may be noted that the observed shrinkage and the computed drying with a diffusion constant of .00010 ft.^{2} per day, are in good agreement.

In Fig. 4 are offered theoretical curves showing the average drying of slabs of varying thickness with both faces exposed. The curves apply only for a diffusion constant of .0001 ft.^{2} per day and for continuous exposure at 50 per cent relative humidity. For slabs exposed to natural weather, periodic absorption of moisture should prevent the indefinite continuation of drying that is indicated by the curves.

Fig. 4 gives curves for only a few thicknesses of slabs and for a fixed diffusion constant. Curves for different conditions can be obtained through use of the similitude relation as follows:

$$\frac{K\Theta}{a^2} = \frac{K_1\Theta_1}{a_1^2}$$

Where K is the diffusion constant, Θ is the time in days, and a is the


Evaporable water lost, per cent of total
6 inch slab
12 inch slab
24 inch slab
48 inch slab
Months continuous exposure at 50% rel. humidity

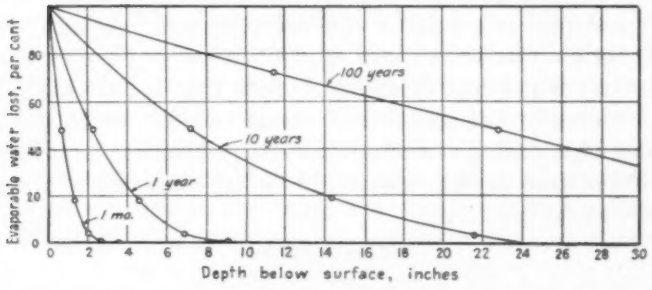


FIG. 4—AVERAGE RATES OF DRYING OF CONCRETE SLABS WITH BOTH FACES EXPOSED TO 50 PER CENT HUMIDITY

FIG. 5—COMPUTED DISTRIBUTION OF DRYING IN MASSIVE CONCRETE STRUCTURES

half-thickness of a slab drying from both faces, or the full thickness of a slab drying from one face. The subscripts are for the different conditions. Thus, suppose a slab has a thickness of 18 in. and a diffusion constant of .0002. Comparing it with a 12-in. slab with a constant of .0001, for which a curve is given in Fig. 4, the relation above indicates that $\Theta = 0.89$. Then if it is desired to know the approximate amount of evaporable water lost in twelve months by the thicker slab of greater diffusivity, the value can be read from the

curve where Θ is .89 x 12 (=10.7) and found to be 41 per cent as compared with a value of 43 per cent for the thinner slab.

The curves of Fig. 4 can also be used for rectangular prisms drying from any number of faces. The manner then is to assume that the prism acts as three slabs, one of thickness equal to the width of the prism, one of thickness equal to the depth, and one of thickness equal to the length of the prism. Instead of determining the amount of evaporable moisture lost, the amount remaining must be determined, merely by subtracting from 100 the values taken from the curves. When the amount remaining is determined for each of the equivalent slabs, these amounts are multiplied together to get the amount of evaporable moisture remaining in the prism. If the prism is drying from four faces only, it is assumed to act as two slabs, and if it is drying from two adjoining faces only, it is also assumed to act as two slabs, but of thicknesses equal to double the width and depth of the prism. The application of computations on slabs to obtain values for prisms by multiplying individual results is mathematically exact and has been described in the references given.

While the curves in Fig. 4 show average degrees of drying, curves are presented in Fig. 5 showing theoretical concentrations of moisture at various depths in a massive concrete structure. Again these curves are only for a diffusion constant of .0001 ft.^{2} per day and for continuous exposure at 50 per cent humidity. Note that the drying penetrates about 3 inches in one month and about two feet in 10 years. It is concluded that drying shrinkage in massive concrete is not important unless the surface drying contributes to the beginning of a crack that otherwise might not form.

APPLICATION

Laboratory tests are necessarily made on small specimens in which drying approaches completion at a given humidity. It has been shown that large members are not likely to dry to the same sextent as laboratory specimens unless subjected to many years of continuous drying. An occasional wetting of the concrete surface will prevent the continued drying over long periods of time, because dry concrete is found to absorb as much moisture in one day as wet concrete loses in two weeks of drying. Therefore, in concrete exposed to weather, the length of the dry season may govern, while in concrete protected from weather, drying may continue indefinitely. Whatever the conditions, the application of diffusion principles permits at least a rough estimate to be made of the shrinkage likely to occur.

If one is interested only in the average drying of a member as a whole, the paper of Glover on "Flow of Heat in Dams"[1] should be

consulted. This paper explains the method and supplies a universal curve for determining the amount of heat (or moisture) remaining in slabs or prisms of any size. The paper of Newman[2] goes farther and gives the solution specifically for drying problems and for cylinders and spheres as well as for rectilinear shapes.

The study of cracking of concrete is beginning to be studied in earnest, however, and for such study the distribution, as well as average amount, of drying in a concrete member is important. Table 1 gives the distribution of moisture in slabs of any size, for any period of continuous drying at 50 per cent humidity, and for any diffusion constant. In order to have one table cover the entire range of conditions, time of drying is given in terms of $K\Theta/a^2$, where K is the diffusion constant, Θ is the time in days, and a is the half thickness of a slab drying from both faces, or the full thickness of a slab drying from one face. The reason for this will be clear when it is realized that any two slabs having the same value of $K\Theta/a^2$ will have exactly similar distributions of moisture.

Suppose it is desired to predict the moisture distribution at the end of a drying season of 100 days in a slab one foot thick drying from one face only. If the diffusion constant is taken to be .0001 ft.^{2} per day $K\Theta/a^2$ becomes .01 and referring to the column in Table 1 headed .01, it is found that 52 per cent of the moisture remains at one tenth the depth from the drying surface, 82 per cent remains at two tenths of the depth, etc. Assuming shrinkage to be proportional to drying, the tendency to shrink should be about 100 — 52 or 48 per cent of the total expected amount at one tenth the depth. If this concrete has the final shrinkage of an average concrete surface cracking is inevitable. A knowledge of the moisture distribution permits estimates to be

TABLE 1—PERCENTAGE OF EVAPORABLE MOISTURE REMAINING IN DRYING SLABS AT VARIOUS DEPTHS FROM DRYING SURFACE

Dist. from Drying Surface	Time of Continuous Drying in Terms of $K\Theta/a^2$														
	.005	.01	.02	.04	.06	.08	.10	.15	.20	.25	.30	.40	.60	.80	1.00
.1 a	70	52	38	27	23	20	18	14	12	10	9	7	4	3	2
.2 a	92	82	67	52	43	38	34	28	24	20	18	13	8	5	3
.3 a	98	93	85	71	60	54	49	41	35	30	27	20	12	7	5
.4 a	99	98	95	84	75	68	63	53	46	39	35	27	16	10	6
.5 a	100	99	99	93	85	79	74	63	55	48	42	33	20	12	8
.6 a	100	100	100	97	91	86	82	72	63	55	49	38	23	14	9
.7 a	100	100	100	99	95	91	88	79	69	61	54	42	26	16	10
.8 a	100	100	100	100	97	94	92	83	73	65	58	45	27	16	10
.9 a	100	100	100	100	98	96	94	85	76	67	60	47	28	17	11
1.0 a	100	100	100	100	99	97	95	86	77	68	61	48	28	17	11

Notes:—K = moisture diffusion constant (.0001 ft.^{2}/day for average concrete)
Θ = time in days
a = half-thickness of slab drying from both faces
For given $K\Theta/a^2$, all slabs will have exactly similar moisture distribution.

made of the depth, size, and spacing of the cracks, but effect of drying is not within the scope of this paper.

It is recognized that the test results and computations presented above apply only for the condition of continuous exposure to a uniform relative humidity of 50 per cent. It is hoped that further studies can be made to determine the effective humidities that prevail under the various exposures that occur in service and for different parts of the country. Laboratory tests should then be extended to include the range of conditions encountered in service.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by April 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for May-June, 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY 1937

A Study of Sub-Aqueous Concrete*

By Arthur R. Anderson†

INTRODUCTION

The deposition of sub-aqueous concrete is usually avoided whenever possible. However, when conditions make the placing of concrete under water necessary, expert supervision is required, and every precaution must be taken to prevent segregation of the aggregates and washing-out of the cement from the concrete.

Two methods of depositing concrete under water are now commonly used. One method uses the tremie tube, which is a pipe of fairly large diameter, and allows the concrete to flow from a hopper above the water surface to the interior of the submerged mass of concrete. The other method employs the drop-bottom bucket, which is lowered into the water with a load of concrete, and when the gates reach the bottom they release the load. Both methods have been successful, but the relative merits of each have been open to question.

PURPOSE

The purpose of this study of sub-aqueous concrete was:

1. To observe and compare the behavior of concrete being deposited under water by: (a) Tremie method; (b) Drop-bottom bucket method.
2. To compare the strengths of the tremie concrete with similar concrete deposited in air under standard control conditions.
3. To find the most advantageous mixture for sub-aqueous concrete as controlled by: (a) Grading of aggregates; (b) Quality of cement; (c) Workability.

SCOPE

A preliminary study included the making of about 60 standard 6-in. by 12-in. test cylinders and 26 10-in. cubical blocks. A number of

*Received by the Institute January 6, 1937.
†Assistant, Civil Engineering Department, Massachusetts Institute of Technology, Cambridge.

TABLE 1—SHOWING AGGREGATE GRADING

Screen Size	Coarse Gradation		Medium Gradation		Sandy Gradation	
	Per Cent Retained	Cumulative Per Cent	Per Cent Retained	Cumulative Per Cent	Per Cent Retained	Cumulative Per Cent
¾ in.	11.2	11.2	9.7	9.7	8.0	8.0
⅜ in.	39.2	50.4	33.9	43.6	28.0	36.0
No. 4	13.6	64.0	12.9	56.5	12.1	48.1
8	8.6	72.6	9.0	65.5	9.3	57.4
14	5.2	77.8	6.6	72.1	8.0	65.4
28	4.2	82.0	5.4	77.5	6.6	72.0
48	8.7	90.7	11.0	88.5	13.5	85.5
100	6.8	97.5	8.6	97.1	10.6	96.1
Fineness Modulus		5.462		5.105		4.685

the cylinders were deposited under water by the tremie method. Other cylinders were made in air according to standard laboratory practice, and used as controls for the specimens deposited under water. The cubes were deposited under water by tremie and by bucket in order to compare the two methods.

After completing the preliminary studies, a series of 10-in. cubical blocks was made to study the effects of all of the variables under consideration. The water-cement ratio was allowed to vary so as to maintain a constant slump of from four to five inches. The cement content was held constant at about 1.5 bbl. per cy. The percentage of sand in the combined aggregate was varied to give different degrees of coarseness to the concrete. The fineness moduli used were 5.5, 5.1, and 4.7, the first giving about the maximum strength for concrete not placed under water, the second being an average for good practice, and the third being definitely over-sanded for ordinary work. Table 1 gives the mechanical analyses for the combined aggregates of the series.

For each fineness modulus, four cubes were made, two of which contained standard portland cement. The other two contained a high early strength cement (made by intergrinding a normal cement clinker with a dispersing agent called T. D. A.). Further comparisons were then obtained by making half the cubes by the tremie method and half by the drop-bottom bucket method.

Only fresh water was used in this study, as the mechanical action of deposition by tremie and bucket was the primary object of investigation, rather than the chemical action of the water.

METHOD

The method of making the test specimens under water required a glass-walled tank in which the action of the concrete was to be observed. The concrete was mixed by hand, and a control cylinder as well as a tremie-deposited sub-aqueous cylinder was made from each batch. Likewise, to compare the tremie and the bucket, one cube was made by each method from the same batch of concrete.

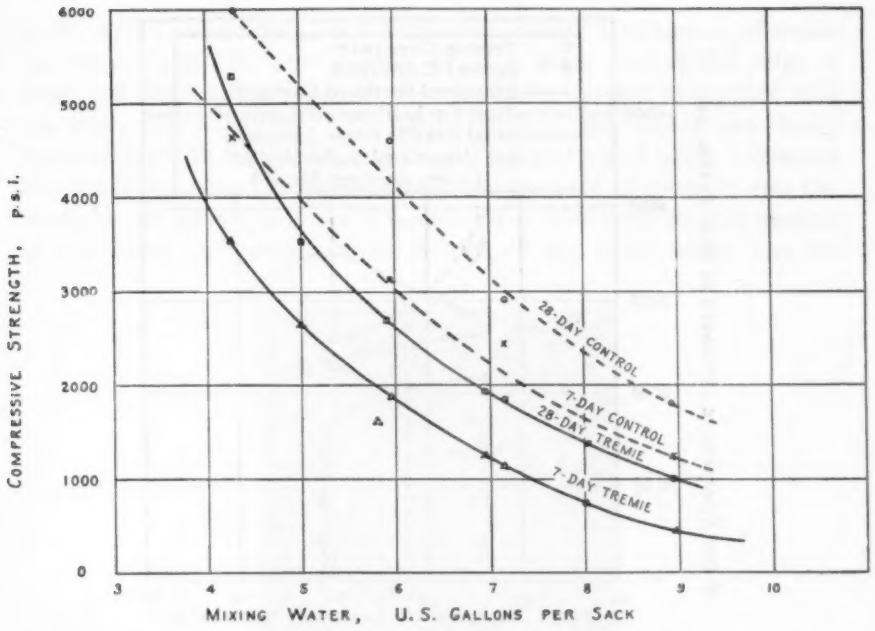


FIG. 1—RELATIVE STRENGTHS OF TREMIE CONCRETE AND CONTROL CONCRETE

It was hoped that in placing the concrete under water the operations would be visible through the glass walls of the tank, but, owing to the clouding of the water by the cement, the action of the bucket was obscured completely. However, in the case of the tremie tube, deposition of the concrete was slightly visible, and the formation of laitance over the concrete was readily observed. (Fig. 4.)

The cubes were cast in a wooden form which could be reassembled after removing the concrete blocks of a previous set. After deposition, the cubes were kept submerged for seven days, at which time they were removed and tested.

In depositing by the tremie method, the tube was completely filled with concrete and plugged before immersing into the tank. After the concrete started to flow into the form, the end of the tube was kept inside the submerged concrete mass, thereby effecting a seal and minimizing the washing effect of the water.

The bucket was filled level full and slowly lowered into position, care being taken not to open the bottom gates until reaching the submerged concrete mass. It was impossible to eliminate completely the

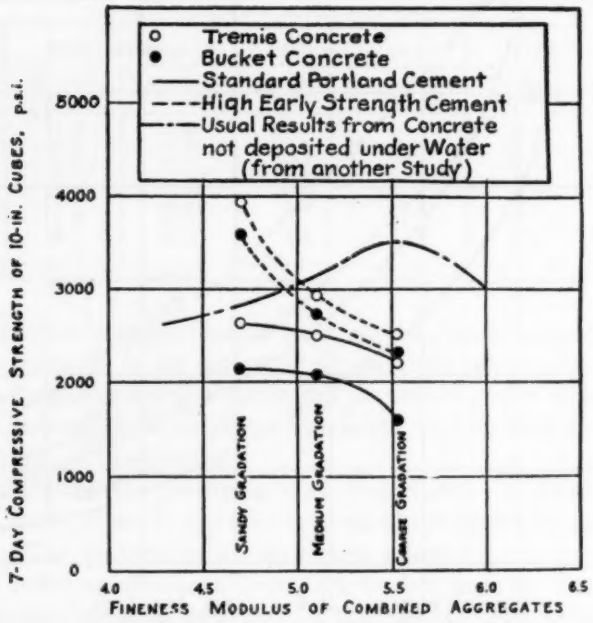


FIG. 2—EFFECT OF GRADING OF AGGREGATES AND FINENESS OF CEMENT ON COMPRESSIVE STRENGTH

washing effect in opening the bottom gates, as their movement caused a local agitation which could be observed by the clouds of cement in the water.

The 6-in. by 12-in. cylinder specimens were tested in a 200,000-lb. hydraulic machine, and the 10-in. cubes were tested in a 1,000,000-lb. Amsler hydraulic machine. The fractured specimens were examined for character of fracture, volume of voids, and texture of concrete.

RESULTS

The results of the tests showed that sub-aqueous concrete at its best has less strength and is probably less durable than the same concrete deposited out of water under standard conditions. Sub-aqueous concrete placed with great care was found to have compressive strengths of from 40 per cent to 80 per cent of similar concrete deposited out of water. The higher percentages were for the richer mixtures. These comparisons can best be seen from the curves of Fig. 1.

Concrete deposited under water by the tremie was on an average 17 per cent stronger than the same concrete deposited by the drop-bottom bucket. The tremie tube allowed the concrete to flow smoothly

into a sealed submerged mass of concrete, where a minimum of washing took place. As the concrete left the lower end of the tube, it displaced the concrete above it, and the surface layers in contact with the water rose uniformly. On the other hand, the bucket was slowly lowered down to the submerged mass of concrete, and when it touched the top of the mass the gates opened and the load of concrete was discharged. A certain amount of washing was inevitable by this method of deposition on account of the motion of the gates which was de-

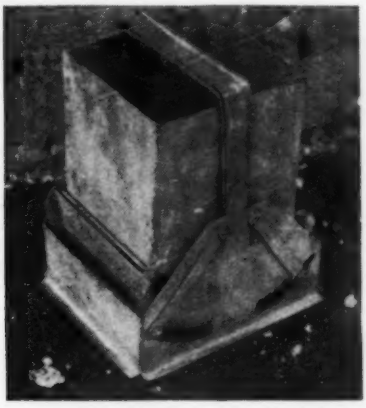


FIG. 3—MODEL DROP-BOTTOM BUCKET, WHICH MEASURES 5 IN. BY 5 IN. BY 5 IN. BOTTOM GATES ARE IN THE OPEN POSITION

tected by the heavy clouding of the water around the bucket. It was found that more laitance was formed on top of the bucket-deposited cubes. This means that more of the cement was washed out of the concrete. Hence the amount of laitance formed gave a measure of the quality of the concrete: the less laitance, the better the concrete.

The relationship between the compressive strength of the 10-in. cubes and the percentage of sand is best indicated by the curves of Fig. 2. The highest percentage of sand (fineness modulus 4.7) gave the highest strength in every specimen of sub-aqueous concrete, while the lowest percentage of sand gave the lowest strength in every case. This differs from the usual results obtained from concrete not deposited under water, for which a typical curve has been included in Fig. 2. The extra sand (which was well graded) gave additional workability to the concrete, decreasing the segregation and washing-out of the cement from the concrete during deposition. Again, the tremie method gave consistently higher strengths of concrete than did the bucket.

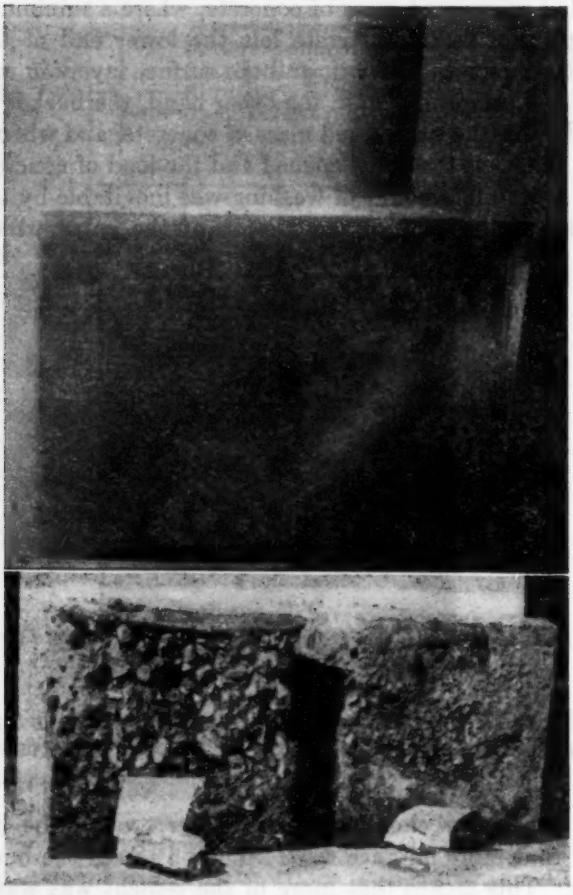


FIG. 4—LOOKING INTO THE GLASS-WALLED TANK. MODEL TREMIE TUBE, THREE INCHES IN DIAMETER, DEPOSITING CONCRETE UNDER WATER. NOTE THE LAITANCE WHICH IS FLOWING OVER THE EDGE OF THE FORM

FIG. 5—TEN-INCH CUBES, SHOWING THE RESULTS OF CARELESS DEPOSITION. THE SPECIMEN ON THE LEFT IS COVERED BY A HEAVY LAYER OF LAITANCE

A finely ground cement with the T. D. A. dispersing agent gave remarkable increases in strength over the standard portland cement in the sub-aqueous concrete. This can be seen from the dotted curves in Fig. 2. For tremie concrete the increase in strength averaged 32 per cent, and for the drop-bottom bucket the increased strength

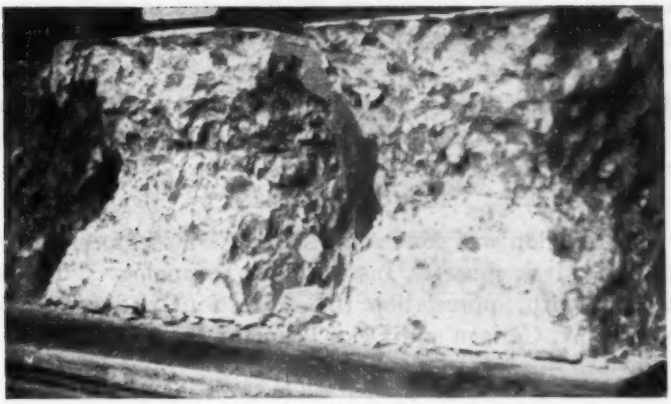


FIG. 6—FRACTURED SPECIMENS WHICH WERE CAREFULLY DEPOSITED UNDER WATER. NOTE THE HOMOGENEOUS AND DENSE TEXTURE

averaged 47 per cent. In appearance, the concrete containing T. D. A. was more sticky and plastic than the concrete using standard portland cement. This "stickiness" had a tendency to hold the concrete together during deposition and to decrease the washing of the cement.

It appeared that the mechanics of deposition was the most important factor affecting the quality of concrete placed under water. A carefully designed concrete mixture was found to be completely destroyed by careless deposition. The aggregates were segregated, and the cement was washed out of the concrete, going into suspension in the water, leaving a mass of sand and stones at the bottom covered by a layer of laitance.

CONCLUSIONS

Although the quantitative results given in this study are based on small scale experiments, it is believed that they indicate at least qualitatively the results to be expected in actual sub-aqueous concrete construction work. The following conclusions can be drawn:

1. The strengths obtained from sub-aqueous concrete were lower than the strengths obtained from the same concrete not deposited under water. In general, the greatest difference in strength was for lean mixes and high water cement ratios.

2. Mixes that would have been over-sanded for concrete placed in dry forms proved to give stronger sub-aqueous concrete.

3. On an average, tremie concrete was 17 per cent stronger than concrete placed with a drop-bottom bucket.

4. The use of finely ground cement exhibited marked increases in strength over that obtained with standard portland cement for concrete cast under water.

5. By employing proper skill and supervision, concrete can be placed under water which approaches in quality concrete cast in dry forms.

ACKNOWLEDGEMENT

This investigation was conducted in the Civil Engineering Department at the Massachusetts Institute of Technology. The author acknowledges with appreciation the suggestions and cooperation of Professors R. W. Carlson and Dean Peabody Jr.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by April 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for May-June, 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MAY-JUNE 1937

Discussion of a paper by Arthur R. Anderson

A Study of Sub-Aqueous Concrete*

T. E. STANTON†
(BY LETTER)

It is rather difficult to conduct laboratory studies of the nature described which will parallel field conditions accurately.

Our experience with subaqueous concrete construction on the San Francisco-Oakland Bay Bridge is rather fully set forth in two papers in the A. C. I. *Proceedings* for 1936, (Vol. 32). In the first, (page 1, of the *Proceedings*) I described the general control problems and in the second, (p. 365) Stanley M. Hands discusses underwater concrete mixtures and placement in some detail.

It will be noted that there was little difficulty in securing a high quality concrete by either the bottom dump bucket or the tremie method.

We felt that it was advisable to use a rather high sand content and high slump in order to insure a blending of successive batches and a flow into the most remote corners not readily reached with the bucket or tremie. One thing to be taken into consideration is the fact that in actual construction there is considerable height and weight of concrete at the point of deposit, thereby insuring sufficient pressure to close up all holes and voids at a lower level.

With the placement tools used, there was very little laitance on the top of the concrete seals even in depths up to 100 feet of concrete as in the case of Pier W-2 where but 2 or 3 in. of laitance was found upon dewatering and the compressive strengths of the cores cut from the top of the underwater concrete (for six sack concrete with a W/C. of 0.85) were:

1906 p.s.i. at 10 days
3562 p.s.i. at 34 days
4313 p.s.i. at 60 days

*JOURNAL, Amer. Concrete Inst., Jan.-Feb., 1937; *Proceedings* Vol. 33, p. 339.
†Materials and Research Engineer, Division of Highways, State of California, Sacramento.

The cores cut from a piece of concrete broken from the seal of Pier W-6 at a depth of 170 ft. below water tested 3954 p.s.i. at 34 days.

These strengths compare very favorably with the field cylinders cast and tested during the progress of the work:

630 specs. 6 sk. struc. conc. 0.85 W/C ratio averaged 4116 p.s.i. at 28 da.
497 specs. 6 sk. underwater conc. 0.85 W/C ratio averaged 3772 p.s.i. at 28 da.

The higher sand content (50 per cent) of the underwater concrete probably accounts for the lower strength of this mix as compared with the structural concrete with the same cement content.

These strengths are somewhat higher than the strengths indicated in Table 1 of the author's paper for the same water cement ratio, a water cement ratio of 0.85 by volume of cement to water being the equivalent of approximately 6.37 gal. of water per sack of cement.

For this quantity of mixing water the author shows a compressive strength approximately 3800 p.s.i. for the 28 day control specimens and 2400 p.s.i. for the 28 day tremie concrete or 65 per cent of the control concrete, whereas, on the Bay Bridge we had an average strength of 3722 p.s.i. for the field cylinders and 3562 to 3954 p.s.i. at 34 days for the cores cut from the same concrete placed under water. Setting temperatures may have had something to do with the strength of the subaqueous concrete as we found that even within the top 12 in to 24 in., temperatures well in excess of 100° F. were developed and at three feet below the surface of a concrete seal, 28 feet under water, the peak temperature reached 129° F.

I cannot, therefore, concur in any conclusion that strengths of subaqueous concrete are necessarily substantially, if any, lower than the same concrete not placed under water provided proper placing tools and placing methods are used.

AUTHOR'S CLOSURE

One may note with interest in Mr. Stanton's discussion of the use of a higher sand content for the subaqueous concrete construction of the San Francisco-Oakland Bay Bridge. This is in agreement with the writer's conclusion No. 3, namely: "Mixes that would have been over-sanded for concrete placed in dry forms proved to give a stronger subaqueous concrete."

As to strengths of cores cut from the under-water concrete of the Bay Bridge, Mr. Stanton calls attention to the high curing temperatures of 100° F. to 129° F. Since the field control cylinders were not subjected to the same conditions of curing as were the cores, it seems that their relative strengths should not be directly compared with the relative strengths obtained in the writer's tests, where standard curing

temperature of 70° F. prevailed for both subaqueous and control specimens.

Furthermore, if the cores are cut from the interior of a large mass of subaqueous concrete, specimens may be taken from a portion of the concrete which was completely free from any disturbance during deposition. These specimens should develop high strength. On the other hand, the writer's specimens, each of which was made up of several miniature loads of concrete, were tested as a whole to reveal all of the defects of deposition.

Even taking into account the difference in curing temperatures and the fact that the cores do not test the entire structure, Mr. Stanton shows by the results of his tests that better strengths are obtained for subaqueous concrete deposited in large masses. Perhaps this is due to the greater ratio of volume of concrete to area exposed during deposition.

In closing, from the data presented on the Bay Bridge concrete, the writer notes with interest the verification of his conclusion No. 5:

"By employing proper skill and supervision, concrete can be placed under water which approaches in quality concrete cast in dry forms."

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY 1937

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers.

The rate of charging tube mills

D. STEINER, *Tonindustrie Zeitung*, Vol. 60, No. 80, p. 990-2, Oct. 5, 1936.
Reviewed by A. E. BEITLICH

A calculation of the charges of balls or cylpeps for various tube mill designs with a discussion of such factors as grinding time, fineness and load of raw materials and dust separation.

Sulfates in cement

Tonindustrie Zeitung, Vol. 60, No. 87, p. 1076-7, Oct. 29, 1936. Reviewed by A. E. BEITLICH

A brief discussion of the studies by Mussgnug on the role of sulfates and sulfides in clinker and cement and their effect on the setting time. The temperature of burning and the atmosphere in the kiln have a pronounced effect on the SO_3 content of the clinker. This may replace partly or completely the necessary amount of gypsum as retarding agent.

Proportion of the mix and cement content of mortar and concrete

Tonindustrie Zeitung, Vol. 60, No. 90, p. 1107-9, Nov. 9, 1936 Reviewed by A. E. BEITLICH

Proposed German specifications giving definition of proportions of mix, and describing methods for the determination of the proportions in hardened mortar and concrete containing aggregates which are (1) insoluble, (2) partly soluble and (3) completely soluble in hydrochloric acid.

New proposed methods for cement burning

F. LIFINSKI, *Tonindustrie Zeitung*, Vol. 60, No. 74 and 75, p. 917-20, 931-3, Sept. 14 and 17, 1936.
Reviewed by A. E. BEITLICH

A discussion of new methods of cement burning proposed or patented by Andreas, Meier, Demisch, Davis, Schroeder, Roemelt and others. The main feature of the processes is the burning of the raw materials suspended in or carried by the stream of the combustion gases. The various types of kilns, the preparation of the raw materials and the economic advantages are demonstrated.

Los Angeles rattler test

L. J. ROTHGERY, *Rock Products*, Vol. XXXIX, No. 12, Dec. 1936, pp. 42-45.
Reviewed by ROY N. YOUNG

The advantages of the Los Angeles Rattler Test as compared with the Deval Abrasion Test and Circular Track Test are discussed. A comparison of results of

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . JANUARY-FEBRUARY 1937

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers.

The rate of charging tube mills

D. STEINER, *Tonindustrie Zeitung*, Vol. 60, No. 80, p. 990-2, Oct. 5, 1936.
Reviewed by A. E. BEITLICH

A calculation of the charges of balls or cylpeps for various tube mill designs with a discussion of such factors as grinding time, fineness and load of raw materials and dust separation.

Sulfates in cement

Tonindustrie Zeitung, Vol. 60, No. 87, p. 1076-7, Oct. 29, 1936. Reviewed by A. E. BEITLICH

A brief discussion of the studies by Mussgnug on the role of sulfates and sulfides in clinker and cement and their effect on the setting time. The temperature of burning and the atmosphere in the kiln have a pronounced effect on the SO_3 content of the clinker. This may replace partly or completely the necessary amount of gypsum as retarding agent.

Proportion of the mix and cement content of mortar and concrete

Tonindustrie Zeitung, Vol. 60, No. 90, p. 1107-9, Nov. 9, 1936 Reviewed by A. E. BEITLICH

Proposed German specifications giving definition of proportions of mix, and describing methods for the determination of the proportions in hardened mortar and concrete containing aggregates which are (1) insoluble, (2) partly soluble and (3) completely soluble in hydrochloric acid.

New proposed methods for cement burning

F. LIFINSKI, *Tonindustrie Zeitung*, Vol. 60, No. 74 and 75, p. 917-20, 931-3, Sept. 14 and 17, 1936.
Reviewed by A. E. BEITLICH

A discussion of new methods of cement burning proposed or patented by Andreas, Meier, Demisch, Davis, Schroeder, Roemelt and others. The main feature of the processes is the burning of the raw materials suspended in or carried by the stream of the combustion gases. The various types of kilns, the preparation of the raw materials and the economic advantages are demonstrated.

Los Angeles rattler test

L. J. ROTHGERY, *Rock Products*, Vol. XXXIX, No. 12, Dec. 1936, pp. 42-45.
Reviewed by ROY N. YOUNG

The advantages of the Los Angeles Rattler Test as compared with the Deval Abrasion Test and Circular Track Test are discussed. A comparison of results of

tests of a number of typical aggregates indicates that the Los Angeles Rattler Test results are concordant with field conditions as represented by the Circular Track Test, whereas the results of the Deval Tests are not.

Preparation of artificial puzzolanic materials

CZERNIN, *Tonindustrie Zeitung*, Vol. 60, No. 96, p. 1183-5, Nov. 30, 1936.
Reviewed by A. E. BEITLICH

A review of recent publications on the preparation of puzzolanic materials by suitable heat treatments. C. Vittori improved the activity of puzzolanic materials by heating them in an electric muffle furnace at various temperatures up to 900° C. The best results in general were obtained with heat treatments at 700°; heating to 900° lowered the activity again. The activity was determined by making compressive strength tests with 2 by 2 in. cylinders made from mixtures of 85 per cent puzzolana and 15 per cent hydrated lime. Reference is also made to the studies by Davis, Carlson, Kelly and Troxell.

Wind pressure on buildings

J. O. V. IRMINGER AND CHR. NOKKENTVED, *Ingeniowidenskatelige Skrifter*, No. 42, 1936.
Reviewed by INGE LYSE

This comprehensive report of experiments conducted at the Royal Technical College of Copenhagen is a most worthy contribution to engineering science. The experiments were made on model of structures in the wind tunnel of the College laboratory and covered such subjects as pressure distribution at sharp edges and their neighborhood, stream flow conditions for bodies placed on the ground and bodies freely exposed to the air current, variation in the wind-pressure distribution on sharp-edged bodies, solid and perforated screens, the sheltering effect of screens, open sheds and partly open buildings. Finally a mathematical analysis is made of the stream field about a building. The report contains 88 pages with several tables and 87 figures.

Oil well cements

S. L. MEYERS, *Rock Products*, Vol. XXXIX, No. 12, Dec. 1936, pp. 54-56.
Reviewed by ROY N. YOUNG

Studies were made of the effects of certain conditions simulating those occurring in the cementing of oil wells as affecting the properties of cement grouts. At 70° F. neat cement under 2,000 pounds pressure gave higher tensile strengths at 24 hours but lower strengths at later periods, than obtained under normal pressure. Compression tests gave similar results. An increase in temperature to 170° F. increased the strengths of the cement grouts at all ages. The setting times were shortened only slightly by the higher pressure, but higher temperatures resulted in considerable acceleration.

In general the properties of the cement grouts would appear to be affected much more by the higher temperatures than by the higher pressures experienced in cementing oil wells.

The specific surface of cements

WOLFGANG HORNKE. *Tonindustrie Zeitung*, Vol. 60, No. 74 and 83, p. 911-4, 1030, Sept. 14 and Oct. 15, 1936.
Reviewed by A. E. BEITLICH

Experiments were made with an optical method for the determination of particle size distribution, and the specific surface of cements was characterized by a certain value. The latter was compared with the results obtained by the usual methods for grain size determination after calculating them to specific surfaces. In the sedimentation apparatus by Andreasen, the height of fall was increased which pro-

longed the fall periods and which in turn made it possible to conduct the determination with alcohol and to determine the coarser fractions too. The procedures of relatively simple and rapid optical methods are illustrated. Elsner v. Gronow discusses the author's views and points out the danger of obtaining quite different results in the interpretations of the specific surface from particle size distribution measurements.

The experimental flat slab at Baku

M. STEUERMANN, *Beton und Eisen*, No 21 and 22, Nov. 5 and 20, 1936. Reviewed by INGE LYSE

A full-size experimental flat slab was constructed by the Russian Construction Trust for the study of: (1) possible savings in reinforcement; (2) dependability of new method for designing flat slabs; (3) possibility of stopping the reinforcement at points of inflection instead of carrying through to support; (4) moment distribution in the external panel where beam support was used.

The slab had uniform panel spans of about 16½ ft. and thickness of 6¼ in. The columns were rectangular of about 21½ x 21½ in. with a height of 13 ft. The load was applied by means of water tanks with no bottom. In addition to deflection measurements, strains in the steel were observed by means of Huggenberger tensometers and the rotation of the column head by means of clinometers. The experiments were very successful and gave valuable information for the design of flat slabs. The author's new method of dimensioning flat slabs was substantiated and the results also indicated that the reinforcement may well stop at the point of inflection.

History of the cement industry in India

Special Publication of The Associated Cement Companies, Ltd., Bombay, marking the merger of the 12 cement manufacturing companies in India, Aug. 1, 1936. Reviewed by J. C. PEARSON

The publication is given over mainly to a brief history of cement manufacture in the country and to a statement of the reasons for a merger and its objectives. Cement was first made at Madras in 1904, but the industry was really started in 1912. During and immediately after the war the industry was prosperous, but during the early twenties, no less than seven new companies were organized. Overproduction led to price wars and failures, a Tariff Board investigation, and the formation of The Indian Cement Manufacturers Association and the Concrete Association of India. In 1930 the Cement Marketing Co. of India was formed to take over the sale and distribution of the product, and this, together with propaganda and education, brought a state of comparative prosperity. However, the quota system on which the C. M. C. operated was found to result in uneconomical distribution, and it gradually became evident that complete fusion of manufacture and sales was necessary to bring the price down within reach of every one. Thus came about the merger, the object of which is declared to be, not monopolistic, but rather to produce cement at the lowest possible cost in India, and to protect the Associated Companies from internal or foreign competition. An extension of the marketing organization and free technical advice and assistance from the engineers of the Concrete Association is planned.

Rapid production of pre-cast concrete units

Cement and Cement Manufacture, Vol. 9, No. 11, Nov. 1936, pp. 233-238. Also *Concrete Bldg. & Concrete Products*, Vol. 11, No. 12, Dec. 1936, pp. 237-243. Reviewed by J. C. PEARSON

A new process by E. Mopin, well-known French concrete engineer, for making precast concrete units by means of vibration, is described in these English journals. The disadvantages of dry tamped and wet cast reinforced concrete as ordinarily

made are reviewed, and the advantages of the new process are set forth at length. While reference is made to the "gelatinous" state of the vibrated concrete, which allows a freshly cast unit to be immediately freed from the mold, there is no evidence of any new property in the concrete that is not common to all concrete that is proportioned for vibration, and properly placed. What seems new, or at least not generally recognized, is that vibrated concrete of suitable consistency (0 to ½ in. slump) has a fairly high degree of cohesion, such that a slab, beam, or even more complicated shapes, may be freed immediately by inverting the molds on flat surfaces. The true shape of such castings is retained, and the concrete has all the advantages of compact, low water-ratio mixtures. If the casting should be injured in freeing the mold, it is easily repaired while the concrete is still plastic, and the molds are of course planned to some extent so that cores, openings, holes for inserts, etc., are easily provided. The process evidently makes for speed, and no cost need be spared on the few molds required, because good molds facilitate the work, and represent only a small outlay as compared with that of the large number of molds required by the usual wet cast process. A special brand of form oil is said to be used —generally a mixture of gas-oil and linseed oil, depending on the kind of mold surface—hard wood, soft wood, or metal.

The article in the products magazine is well illustrated, showing examples of exposed aggregate panels, grill work, ornamental fencing, poles for power lines and examples of the large housing developments undertaken in France, based on this system. Projects amounting to several millions of dollars are under way or have been completed.

The auscultation of reinforced concrete precast piles

MARC LACOMBE, *LeGenie Civil*, Vol. CIX, No. 13, Sept. 26, 1936, p. 263-265.

Reviewed by R. L. BERTIN

This article describes a simple and effective method of detecting, during the process of driving precast reinforced concrete piles, any injury such as cracks, breaks or overstress resulting thereto from obstacles encountered in heterogeneous soils.

The method consists in providing one or more longitudinal conduits within the pile having side connections near its top and bottom. The conduits are formed by means of porous tubing, the modulus of elasticity of which is less than that of the concrete.

Prior to the pile being placed in the leads, the upper end of the conduits is connected by means of a flexible hose to a pressure pump and a liquid pumped into the conduits. When the conduits are air free, the bottom connections are closed and the liquid subjected to a pressure of about 2 kg. per sq. cm., depending on the porosity of the concrete. It requires from 15 to 30 minutes before this pressure becomes approximately constant.

The pile is then placed in the leads and driven in the ordinary way, the hose connection being maintained during driving.

If during driving, the pile is not subjected to abnormal deformation the pressure, indicated by a pressure gauge, will remain constant. If, on the other hand, the pile is strained excessively and slight cracks develop, the pressure will drop slightly and if the driving is continued the pressure will fall suddenly at the time of breaking.

To determine the extent of cracks, as soon as a slight drop of pressure is noticed, the pressure is brought back to its original value and the time required for a given

pressure drop is recorded and compared with previously prepared charts from which the extent of the damage is determined.

Observations of piles driven and pulled are given showing that the rate of penetration does not indicate abnormal conditions. These piles, the rates of penetration of which were practically identical, when pulled showed that one was in perfect condition, another was broken in three sections and the third was completely cracked.

Tests on shrinkage of concrete

J. R. SHANK, Ohio State University, *Engineering Experiment Station News*, Vol. VIII, No. 4, Oct. 1936, p. 6. Reviewed by J. R. SHANK

Shrinkage observations were made on specimens of plain concrete and heavily reinforced concrete in room air for something over a year and a half. The plain specimens were 4 x 4 x 25¼ in. and the reinforced specimens about 15 sq. in. gross cross-section carrying two ⅞-in. round bars. These specimens were control specimens for an investigation of the plastic flow in bond for lap bar splices. Five cement-water ratios by weight 1.2, 1.4, 1.6, 1.8, and 2.0 were used. It was found that a power expression $y = cx^{\frac{1}{2}}$ where y represented the unit shrinkage and x, days, followed the observed shrinkage very closely for about 50 days. The observations were begun 28 days after standard curing at about the first of the year when the room air humidity was low. The fifty days marked the decided increase in humidity at about March first. No marked difference was had for the different cement-water ratios. At 50 days the shrinkage ranged from 0.000415 to 0.000490 in. per in. At 550 days all but one (the c/w = 1.4 which produced the high at 50 days) ranged between 0.000495 and 0.000520 in. per in.

The measurements on the corresponding reinforced specimens were made on the steel by means of a 20 in. strain-gage. The length of embedment was 18 in. A comparison was made between the stresses computed for the steel from the shrinkage data of the plain specimens, considering no cracking or bond slipping or plastic flow, and the observed average stresses. Up to the point of from 13.5 to 21 days, when the cracks apparently occurred, the observed stresses were generally about one-third that of the computed stresses. The observed average stresses at cracking, which were the maximum, ranged from 11 to 23 thousands of p.s.i. At 50 days the average stresses for all but the two leaner mixes held to about 12000 p.s.i. The humidity and temperature contraction and expansions during the spring and summer months eliminated practically all of the stresses in all cases. It should be noted that the theoretical calculated stresses assumed instantaneous bond at the ends of the 18 in. of concrete whereas, if 9 in. were required to develop the bond and the development were uniform, the calculated stress should be twice the observed average stress. In one case, that for the c/w of 1.6 the 9 in. were sufficient to develop full calculated stress.

Cement testing and "special cements"

J. H. JENNINGS, *Cem. & Cem. Manufacture*, Vol. IX, No. 10, Oct., 1936, pp. 221-225. Reviewed by J. C. PEARSON

In a recent article in *Zement*, "Nachteile Unsere Heutigen Zement Normen" Dr. Walter Dyckerhoff deals with two questions of concern to the cement manufacturer, (1) whether the great variety of portland and other cements for special purposes favored by some authorities in the United States is to be a permanent feature in cement manufacture, (2) whether the present standard methods of testing cement are leading cement manufacturers in the right direction to improve the

quality of their product. In regard to the first question Dyckerhoff shows that the tendency toward special cements generally involves the enhancement of certain properties at the sacrifice of other equally important properties, that the need for these special cements is over emphasized and not well established, and that the manufacturer who believes in thorough mixing of raw materials, mixing of clinker and mixing of ground cement to produce a uniform product of high quality cannot maintain such standards of uniformity and quality if his plant has to be shifted from one product to another at short intervals. His answer to the second question is quite emphatically negative, showing by comparative tests of cements according to present methods and the new method proposed by Haegermann (plastic mortar with .6 w/c by weight) that the former are often misleading.

The author is in complete agreement with Dyckerhoff, but he would go even farther. Thus he says that specific surfaces rather than the residue on a standard sieve should be used for fineness, if it is considered necessary to include some such measurement in a standard specification. But, he adds, there seems to be no particular reason why this item should be in the specification at all, because it does not matter to the user what specific surface his cement has, so long as all the other properties are satisfactory.

With reference to the proposed new plastic mortar test in Germany, the writer states that for some years at his own plant he has used 3 in. cubes of concrete containing a maximum sized aggregate of $\frac{3}{8}$ in., and a definite w/c of .66 by weight, as a control on his manufacturing processes. He makes the standard tests as well, because they are required, but the quality control is based on the concrete tests.

(It may be of interest to note that Mr. Jennings tests are in line with the proposed new studies of A. S. T. M. Com. C-1, which are to be undertaken in the very near future.)

Electrical treatment of concrete to prevent freezing and to make concreting possible in winter

C. KUNZ AND E. FONTANELLAZ (Berne) and P. HALLER (Zurich). *LeGenie Civil*, Vol. CIX, No. 9, Aug. 29, 1936, p. 184-187 and No. 10, Sept. 5, 1936, p. 204-207. Reviewed by R. L BERTIN

In 1934 the technical papers of Switzerland reported a method of applying an electric current to concrete to prevent the influence of freezing, called "electro concrete," which had been tried in Sweden. Since then, laboratory tests and practical applications were made in Switzerland. This paper is a report of the results obtained.

The time necessary for the setting of concrete varies with the temperature, being more rapid with high than low ones. An electric current can be passed through fresh concrete and since impure water is a poor conductor the electrical resistance causes a rise in temperature which is additive to that caused by the chemical reaction between water and cement, the latter is insufficient to counteract the action of frost. Once the setting is completed, there is no more water present and the electric current ceases to pass through the concrete. From this principle, two systems are derived: (1) Superficial treatment; (2) Mass treatment.

Superficial Treatment—This method is applicable to floors, slabs and the like. The equipment consists of strips of metal slightly curved crosswise to insure a good contact at their edges and to give them greater stiffness. They are about 15 cm. wide and 1.30 meters long, spaced about 5 cm. apart and connected alternately with the two poles of an alternating current supply. A tension of 40 volts proved sufficient to induce proper setting and sufficient hardening.

Mass Treatment—For walls, strips of metal 7 cm. wide and 1 mm. thick, extending 20 cm. above the top of the forms are nailed to the interior surfaces of the forms. after the forms are filled with concrete the protruding ends of the strips are connected to an electric circuit so that the current on each side of the walls flows through the mass. This method will work efficiently in walls of 70 cm. and even up to 1.30 m. thick. For greater thicknesses, short rods sunk in the fresh concrete 25 cm. apart, 50 to 60 cm. long and 8 to 12 mm. in diameter are connected alternately to an alternating current supply. A number of practical applications are described in this paper. The economical aspect of this process is discussed at length.

The quantity of current required under different conditions and the method of computing it to obtain the desired temperature are treated in great detail.

The use of the reinforcing rods as one of the poles suggested by Mr. Rethy is described. However, the authors warn against the possibility of this method affecting the adhesion between concrete and the steel.

The authors conclude as follows: "Experience shows that concrete treated electrically possesses the same qualities as ordinary concrete. The cost per cubic meter of concrete so treated is rather high and the treatment is therefore limited to its use where concrete work must proceed in winter and the cost is offset by the advantages accruing to the owners in securing their structure without delay."

A practical method of extablishing the composition of concrete

R. Feret, *LeGenie Civil*, Vol. CIX, No. 20, Nov. 14, 1936, p. 431-432. Reviewed by R. L. Bertin

The author states that a general solution of the problem of determining the best proportions of the constituents of concrete has not as yet been found. Many methods on the other hand exist for proportioning concrete for given materials which are more or less applicable on the job.

He quotes as one of the oldest, and favored in America, that based on the laws established by Abrams to the effect that the strength is an exponential function of the water cement ratio. This function, he states, is not easily handled and its application further complicated by the fact that the fineness modulus of the aggregate is involved.

Research at Lehigh University led to the discarding of this method in preference to one in which the direct relation of strength to cement water ratio is used. This property was noted long ago, notably by Bolomey and Dutron and has been the basis of many practical applications in Europe.

The author reviews the method described by George Robert Wernisch in *Concrete* of Dec., 1935, which rests on the laws established by Prof. Inge Lyse, to the effect that with given materials, the slump of a concrete mixture will remain constant so long as the quantity of water added to a given absolute volume of the solids is kept constant. A graph showing the relation of strength to cement water ratio is given for different ages and several examples demonstrating the application of the method are given. Attention is called to the fact that this method is aimed at the determination of proportions for a given cement, fine and coarse aggregate.

The author indicates how this method, although not apparently foreseen by its originators, can be extended to apply to the use of aggregate composed of elements graduated from fine sand to the largest size stone or gravel or for important structures in which it is generally desirable to separate the aggregate in a number of calibrated parts and to recombine them in the most favorable proportion, for which the author advocates the use of the rule established by Bolomey, namely, that the

proportion of elements passing each sieve used must increase in direct proportion to the square root of the diameter of the holes.

To this end he suggests the use of a graph in which the ordinate measures the the proportions in question and the abscissa not the logarithm of the diameters as is usually done, but by the square root of the sieve openings in which case the ideal granulometric curve is a straight line and the operations greatly simplified.

A straight line is so drawn that it expresses in a mixture of cement and aggregate the total proportion, p, of elements passing a given sieve, the size of which is d in terms of the formula established by Bolomey,

$$p = A + (100 - A)\sqrt{\frac{d}{D}}$$

in which A is the proportion of cement (determined by the method referred to above) which must enter in an absolute volume 100 of the dry mixture and D is the largest diameter of the aggregate.

Causes and effects of alteration of cements due to long period of storage prior to their use

LeGenie Civil, Vol. CIX, No. 16, Oct. 17, 1936, p. 340-341. Reviewed by R. L. BERTIN

This is a review of Technical Paper No. 19 (Building Research) published by the Department of Scientific and Industrial Research of London. This paper is a preliminary report of a systematic study of the changes of the properties of portland cement resulting from long periods of storage. The review is limited to the conclusions stated in the paper, as follows:

Causes of Alterations—The alterations result from the absorption of water and carbonic acid from the air. Dry air containing CO_2 has no effect on freshly ground cement. Damp air free of CO_2 has but a feeble effect. For damp air containing CO_2 the loss by ignition and the CO_2 content increase very rapidly with the length of exposure.

Variation of the rate of absorption of CO_2 with time—Tests were made on three portland and one rapid hardening cements to determine the variation of loss by ignition and CO_2 content in per cent of the weathered cement in terms of time. The speed of absorption, approximately proportional to time, was at first high until a critical zone was reached beyond which it became constant but to a lesser degree. These characteristics were different for different cements.

Influence of the carbonation of cement on its tensile resistance—Tests were made at 1, 3, 7, 14 and 28 days on specimens of neat cement and mortar. For the neat cements, the resistance decreases in proportion to the degree of carbonization up to a certain critical CO_2 content reached at the three-day period. Beyond seven days this critical content seems to have no effect on the strength until a second critical CO_2 content is reached, beyond which the strength tends rapidly towards zero.

The tests on mortars differ from the above in that (1) weak carbonation has no effect on the strength, and (2) the resistance continues to increase for high degree carbonation at which it has stopped for neat cement specimens. The existence of a critical zone shortly after the set indicates that the reactions between the cement constituents and the calcium carbonate are not the same before and after the critical zone.

Influence of Carbonation of Cement on the time of set—The tests on the cements and also their constituents created during the setting have not furnished very definite results, except that where the carbonate contents are high the set begins suddenly.

This is accounted for by the relative thickness of the film of calcium carbonate covering the cement particles. If the film is thick it cracks under the presence of water and allows the water to act on the internal parts of the particles hence the acceleration of set.

Influence of the aggregate—Three series of tests were carried out to seek the cause of the different behavior of neat cement and mortar mentioned above, insofar as tensile resistance is concerned. The water cement ratio strength relation does not apply to mortars. For aggregates composed of one sized particles, the resistance becomes appreciable only when a certain size particle is used; from then on the resistance increases as the size of the sand particles are increased until a maximum is reached beyond which the resistance slowly decreases. With the sand particle size giving the highest strength, and varying the cement to sand ratio the resistance becomes appreciable when at least 50 per cent of sand is used and passes through a maximum for about 75 per cent of sand.

Lightweight concrete

Bulletin Research Bulletin, No 15, Department of Scientific and Industrial Research, London.
From BUILDING RESEARCH SUMMARY

The demand for lightweight concrete in modern building is increasing and several novel materials can be used for making it, in place of pumice mainly imported from the volcanic deposits near Coblenz in Germany, or furnace clinker and coke breeze. One of these is "foamed slag" made by rapidly chilling molten slag from blast furnaces manufacturing pig iron. Foamed slag is extensively used in Germany and competes seriously with pumice even in the vicinity of the pumice quarries. Building blocks are made of various sizes and shapes so that it is not necessary to cut blocks during construction. It is claimed in Germany that blocks 50 per cent larger than ordinary clay bricks are only half the weight, and for the same volume are only half the price of clay bricks. An example in this country (England) of the use of foamed slag concrete blocks is to be found in the Fire Testing Station at Elstree, Hertfordshire, recently erected for the Fire Offices Committee.

Production has also been begun in this country of a light material obtained by rapidly heating clays and shales, while the Building Research Station has found that certain slates when heated expand to many times their original thickness. The resulting concrete is cellular in structure and will float on water.

A table giving figures for the properties of various lightweight concretes shows that those made from foamed slag and expanded slate compare very favourably with those made from other materials.

Lightweight concretes have been used mainly for internal work. As regards external work various failures that have occurred in Great Britain owing to lack of knowledge of the properties of the concretes have prevented a more extended use of these materials in external walling. "It is felt, nevertheless, that any consequent general restriction of the use of lightweight concretes is unnecessary, since much fuller information on the properties of these materials and the precautions necessary in their use is now available. Lightweight slag aggregates and pumice have been extensively and most successfully used in Germany in external wall construction, while in France lightweight concretes are freely used for the inner leaf of hollow walls in framed structures. In view of the great economies in weight and the enhanced thermal insulation afforded by walls of this kind, it would be unfortunate if their rational development were to be prejudiced."

For many of the purposes for which lightweight concretes are required, a very modest mechanical strength is adequate. The strength of concrete increases with

the proportion of cement used in it but the expansion and contraction of the concrete as it picks up and loses moisture with changing weather conditions also increases and this may cause cracks and other troubles. Mixes with only 1 part of cement to 12 parts of lightweight material are used for panel fillings for external walls in Germany and the use of such lean mixes may be an important factor contributing to the successful work which has been carried out abroad in lightweight concretes.

Porous concretes cannot be expected to afford any high degree of protection to embedded steel from external corrosive agencies, so that some additional protection will be necessary.

Grouting Boulder Dam tunnels

V. L. MINEAR, *Civil Engineering*, Nov. 1936, Vol. 6, No. 11, p.. 751. Reviewed by J. R. SHANK

Approximately 235,000 cu. ft. of cement for grout, or more than 50 per cent of of the total amount placed at Boulder Dam, was used in grouting 22 of the tunnels and their appurtenant works. Eighty per cent of this grout was injected through the walls of the 50 ft. diversion tunnels.

The grout-placing equipment consisted of a mixer, a mechanically agitated sump, two mud pumps and a high pressure hose. Mixing water was controlled by a disk-type meter with a 6-in. vertical dial, having scales reading to tenths of a cubic foot and a totalizer. An automatic pressure regulator controlled the power air supply. Trouble was had when a continuous operation took several hours due to the partial setting of cement in the pump valves. This was obviated by learning by the sound of the exhaust and the general behavior when fouling was taking place when water was circulated to the full capacity of the pump for about one minute which washed out the incipiently hardened cement.

Unscreened neat cement grout with water-cement ratios varying from 7.0 to 0.55 were used. The amount of cement that can be injected into a given hole, the greater the better, is dependent upon the character of the rock penetrated, the pressure applied, the rate of pumping and the water-cement ratio. The injection of water at high velocity preliminary to grouting opens up passages through the "gouge" or other material deposited by infiltration by carrying it to cavities at a greater distance from the mouth of the hole. It was found that when the rock was dry, water was taken from the grout and the thickened residue plugged the hole preventing proper grouting back of this plug. The initial injections into all high-pressure holes were therefore of plain water or thin grout mixture. Pressures as high as 750 p.s.i. were used. Difficulties due to sedimentation of the cement in the larger spaces due to lessened velocities were overcome by maintaining a reasonably high pumping speed. Close control of the rate of pumping was found necessary as a sudden increase in either speed or pressure frequently resulted in the holes refusing grout shortly thereafter. A pump-man can plug tight (unprofitable) holes by opening and closing the pump by-pass valve in such a way as to cause water hammer. More holes may be lost by inexperienced men attempting to use a thicker grout than the hole will accommodate than from any other single cause. An effort was made to inject the maximum number of sacks per hour. When this rate had fallen to where the hole was approaching refusal, water was injected for five minutes which invariably caused the pumping speed to increase. The criterion for final refusal, indicating a finished hole, was when a hole refused to take more than one cubic foot of grout at the given pressure in ten minutes.

Leaks at shrinkage cracks in the tunnel walls were stopped by chipping with a calking tool the concrete at the edges of the crack and driving the chips into the

crack, followed by the application of a blend of portland cement and quick hardening calcium-aluminate cement, or by driving thin, dry soft pine wedges into the cracks and breaking them off flush. A mixture of 2½ parts portland cement to one part calcium-aluminate cement thoroughly dry mixed was used.

To grout exposed rock surfaces thick grout at low pressures was applied and allowed to harden largely for the purpose of stopping leaks which could not well be stopped by the method used for the tunnel wall. At a later date a second series of holes was drilled and another stage was grouted. This stage grouting was continued until the area to be grouted was covered.

Skrinkless and expanding cements

HENRY LOSSIER, *LeGenie Civil*, Vol. CIX, No. 14, Oct. 3, 1936, p. 285-287.

Reviewed by R. L. BERTIN

The author repeats the definition of shrinkage of concrete hardening in a dry state which he gave in his article "Cracks in Reinforced Concrete," (*LeGenie Civil*, Feb. 22 and 29, 1936—reviewed in A. C. I. JOURNAL, May-June, 1936 issue, page 735) as an apparently very complex phenomenon, resulting partly from irreversible chemical changes and physical and chemico-physical actions of capillary origin, imperfectly reversible under humidifying conditions.

He reviews the various methods which have been propounded for the last forty years to overcome the practical inconvenience of shrinkage, among them prestressing the concrete. More particularly he describes the simple process proposed by Lund and Koenen in 1907, namely, prestressing the reinforcement of members intended to sustain tensile stresses, maintaining the reinforcement under stress until the concrete has hardened, then releasing the stress, thus subjecting the concrete to a compressive stress smaller or greater than or equal to the working stress thus reducing or eliminating the formation of cracks.

This method is applicable to reservoirs, pipes or conduits under pressure, tension members of bow strings, etc. The same results would obtain by using an expanding cement which would place the reinforcing in tension and the concrete itself in compression. Of as much interest would be a cement of reduced or annuled shrinkage for mass work, such as dams, or of large surface as for roads.

For several years, research work with a view of controlling the linear variation of cements have been of no avail because of the instability of the results.

A new angle of approach, as indicated in the author's article of Feb. 29, through the development of a treated cement by Mr. Hendrick, Technical Director of Poliet and Chausson of Paris, presents itself.

The principle at the root of his conception is to compensate the normal shrinkage due to the desiccation of the colloids and of the capillary channels by means of a corresponding increment of the volume of the crystalin skeleton. This development of the latter is obtained by retarding the dissolution of certain anhydrid constituents of the cement and causing the formation of crystals which occupy the volume of water released by the colloids and the capillaries.

Preliminary tests were made with this treated cement with a view of determining: (1) the stability of the expansion with time; (2) the relation between the resistance of specimens made with treated cement and plain cement; (3) same as 2nd for confined specimens.

The resume of the results obtained follows:

(1) *Stability.* Tests on cement paste specimens 3 cm. x 3 cm. x 25 cm., kept damp at a constant temperature of 18° C. showed expansions of the order of 3 to 4 mm. per meter.

(2) *Resistance of Free Specimens.* The structural modification caused by the expansion phenomena cause a reduction in strength. The strength relations of treated cement to ordinary cements are: .51 at 7 days; .62 at 28 days; .74 at 90 days.

(3) *Resistance of Confined Specimens.* Molds so made that the concrete could be confined when fresh were used. Specimens were removed after 24 hours and 7, 28 and 45 days.

The confined specimens showed the following strength increase over those freed at 24 hours: at 28 days 19 per cent; at 45 days 75 per cent.

More complete tests are under way and will be the subject of a later publication.

A statistical study of tension tests of one brand of portland cement

H. WALTER LEAVITT. Maine Technical Experiment Station. Paper No. 18, University of Maine 1936. HIGHWAY RESEARCH ABSTRACTS

The accumulation of data in the State Highway Testing Laboratory at the University of Maine concerning one brand of portland cement used since 1927 suggested a statistical study of its constancy in strength over a period of years to throw some light upon such questions as the following: Is portland cement today similar to that produced a few years ago? Does it have the same relative tensile strength in a standard 1:3 mix with Ottawa sand when tested at the ages of 7 and 28 days? Has the demand for high early strengths lessened the normal increase in strength in the curing period between 7 and 28 days? Have better manufacturing methods, especially finer grinding, produced a difference in the tensile strength of ordinary portland cement? How constant is the laboratory performance or technique maintained?

A 1:3 mix with standard Ottawa sand was used throughout the study, and the per cent of mixing water remained nearly constant throughout any one year, changing but little during the entire study. For the first 7 years one operator performed the major part of all tests. Most of the testing for 1935 was done by one operator, although two other operators made some of the tests. The testing laboratory had no control over the number of samples it received. The only test data included in the study were those of tensile strength; all the tension briquets being fabricated, cured, and broken according to standard A. S. T. M. requirements. For statistical analysis of the data, common methods of simple correlation were used.

TABLE I—STATISTICAL RESULTS ON 7-DAY AND 28-DAY TENSILE STRENGTHS BY YEARS, 1928-35

	1928	1929	1930	1931	1932	1933	1934	1935
No. of samples	76	127	136	145	162	84	56	115
*r (7-day vs. 28-day)	0.454	0.433	0.382	0.503	0.207	0.116	0.409	0.392
Mean 7-day	332.2	341.4	339.0	326.5	336.3	342.3	330.4	352.3
Mean 28-day	408.3	416.6	409.2	418.9	424.0	426.6	427.4	418.7
**S. D. 7-day	29.56	23.32	27.16	17.79	23.78	23.45	16.95	26.9
**S. D. 28-day	23.61	22.06	25.16	15.56	16.68	16.50	16.54	24.7
28-day minus 7-day	76.1	75.2	70.2	92.4	87.7	84.3	97.0	66.4

*r = correlation coefficient.
**S. D. = standard deviation.

Statistical constants in the accompanying table show mean or average strengths for each of the eight years and for each age at time of sampling. The standard deviations are also recorded, as well as the increase in strength between 7- and 28-day tests for each year. The difference between the 7-day performance of the cements tested up to 1930 and those tested since then appears to be the result of

changes made by the manufacturer in the relative silicious and calcareous contents of the cement. In spite of the slightly increased strength at the 7-day age, 28-day strengths remained very constant during the last 5-year period. The difference between the mean 28-day strengths and the mean 7-day strengths also held fairly constant through the period 1931-34, while the corresponding difference for 1935 was noticeably lower because of the marked increase in 7-day strengths. Evidently some improvement in method or some alteration in the chemical ingredients of the cement was responsible for this change. It is believed that the change in laboratory operators in February was responsible to some degree for the higher standard deviations for 1935.

In conclusion, the brand of cement studied has shown a slight increase in strength throughout the eight-year period, the increase in strength between the 7-day and 28-day test periods remaining about the same. Laboratory technique which did not vary greatly during the first 7 years suffered considerably from the change of operators in 1935. Similar statistical analyses of data are recommended for all testing laboratories for the purpose of checking up on relative variance of results.

Grouting the foundations of Boulder Dam

P. A. JONES AND V. L. MINEAR, *Civil Engineering*, Vol. 6, No. 12, Dec. 1936, p. 810
Reviewed by J. R. SHANK

Two other papers on the grouting work for Boulder Dam have been abstracted and appear in the Sept.-Oct. JOURNAL, page 78 and in this issue.

The foundation grouting was divided into three classes, A. B, and C holes. Holes "A" started on the axis of the dam from the lower drainage gallery, were spaced about 5 ft. apart and were driven to as much as 150 ft. depths at angles of 15 degrees upstream from the normal to the gorge bottom and walls. Holes "C" started on top of the upstream toe and sloped 30 degrees downstream toward and to within 20 ft. of the line of "A" holes. Holes "B" were relatively shallow holes, not over 50 ft. deep, spaced about 20 ft. any way over the whole foundation area upstream from the axis.

The grouting of the "B" holes was done before concrete had been placed and was designed to seal major surface faults and fractures so as to provide a leak-proof layer under the up-stream part of the dam and also to provide cover for the high pressure injections in the deeper "A" and "C" holes. The original design contemplated grouting the "A" line in advance of the "C" line. However, it became necessary to reverse this program in order to finish the "C" line before the storage of water in the reservoir began. Hence, after 11 primary holes within the inner gorge on the "A" line had been grouted, this work was temporororily discontinued until all the "C" holes on the canyon floor had been drilled and grouted. Grouting of the "C" holes was done progressively, that is, a hole was drilled and grouted before another was drilled. This procedure was not strictly adhered to in the case of the "A" holes

An interesting feature was a rather elaborate telephone system used by "grout scouts" with test sets who were able to report leaks with a minimum of delay.

A few notes from the experience obtained on this vast work follow:

Pump valves and seats may wear out in a few hours and a stand-by pump is advisable.

Lubricated plug valves are to be preferred over ordinary iron cocks.

Gages are most satisfactorily protected by filling the short gage tube with a semi-fluid waterproof grease.

Frequent flushing of the pump and pipe lines is required. Long pipe lines should be avoided.

Gouge or other material deposited by infiltration should be washed out or ahead before grouting begins. In comparatively tight rock, it is advisable to flush the holes by introducing water at the bottom of the hole through a smaller pipe and washing upward outside the pipe.

All holes should be flushed out immediately preceding the introduction of grout. In broken rock, nearly full grouting pressure should be applied and maintained as long as there is any noticeable "pick-up" or increase in speed.

Use the greatest pressure safely possible. The maximum pressure should not be great enough to disturb the natural formation by lifting parts of the rock from the bedding planes.

The water-cement ratio, pressure, and pumping speed should be coordinated by "feeling out" the hole. The judgment of experienced operators is necessary for this.

Initial injections of grout having a water-cement ratio equal to 0.01S where S is the limiting pumping pressure, worked well.

Two ways of clogging and therefore destroying a high-pressure hole in porous rock are by using too thick a grout and by pumping very slowly.

The pumping speed was controlled by manipulation of the water-cement ratio. When the speed fell below normal (about 15 forward strokes per minute) the water-cement ratio was increased proportionately.

The maximum pressure specified should be obtained by pumping at a constant pressure of from 200 to 300 lb. in excess of the desired static or back pressure of the hole and it was held important to pump at a constant pressure continuously to force the maximum amount of grout into the hole in the shortest possible time.

A neat grout of standard portland cement is probably the best all-purpose grout. The use of rescreened cement increases the difficulty of stopping leaks though in tight rock the use of rescreened cement permits the introduction of as much as 10 per cent more grout.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MARCH-APRIL 1937

President's Address at 33rd Annual Convention

The Institute Carries On

By F. R. McMillan

For the American Concrete Institute, the period of depression seems definitely to have passed. Membership and income are again increasing and the day of the balanced budget with a small margin for surplus is at hand. This is not the occasion for a detailed discussion of the Institute's finances, but now that better times are in sight, it is fitting for the purpose of record that mention be made of the successful management which the Institute has enjoyed during the trying years through which we have just passed. Summary tables will later be published in the Journal which will show how, with falling membership and dwindling income, the Institute kept going and finally came through with some of its reserves still intact. For this successful outcome we are indebted to our Secretary and the loyal members of his staff, who with greatly reduced salaries and increased burdens, carried on effectively and uncomplainingly.

As was to have been expected, the marked reduction in income resulted in the curtailment in the number of issues and size of the Journal. But it also forced a much tighter editorial policy which has definitely raised the standard of our publications. This improvement in the character of papers, together with the addition of the department of current literature reviews in the field of cement and concrete, which also came about during the lean years, have made the Journal of the American Concrete Institute one of the leading engineering publications—a magazine of current interest and high reference value.

These improvements were not accidental. They were the result of the careful planning by the Board of Direction, painstaking work

on the part of the Secretary, and the able and untiring efforts of the Publications Committee, and particularly its Chairman, J. C. Pearson, the President-Elect.

In noting these achievements of the depression, we must not overlook the responsibilities of the future. As our membership expands and our income rises, the JOURNAL must expand accordingly, not by a mere increase in bulk but by a studied increase in the breadth of its coverage. The same tight, or even tighter, editorial policy should be continued with respect to individual contributions, but the range of subjects covered should be increased and new contributors sought out.

A report of our Committee activities is not so favorable. It is true that some excellent committee work has been done during the period of depression. The work of the column investigation committee in obtaining funds and carrying out an epochal series of tests was an outstanding achievement, both in magnitude of effort and importance of results. The building regulations for reinforced concrete (1935-1936) represent committee achievement of great practical significance. Other noteworthy reports have also appeared, but considering the extent of our field the record is not particularly gratifying.

Some consolation for this inactivity may be found in the fact that when other interests of life are in upheaval, it is difficult to concentrate on committee work. To a considerable extent, however, the deficiency in this field has been due to the attitude of the Institute itself.

As pointed out by Past-President Bates in his stimulating analysis of Institute affairs one year ago, the American Concrete Institute needs a more vigorous policy towards the work of its committees. Membership on committees should not be regarded as an honor granted for distinction in a particular field, but as an opportunity for wider service. Acceptance of committee appointments should carry with it an obligation to work. Failure to perform, either on the part of the individual or of a committee, should call for a prompt change. Only by such a policy can effective committee work be built up.

Mr. Bates' admonitions have not gone unheeded. You will see in the future a more intensive committee activity. One development already under way is the stimulation of more committee reports of the "how to do it" kind. One such report is that which will be presented at tomorrow morning's session on the Effect of Plastic Flow and Volume Changes on Design. Another is the Manual of Concrete Inspection now in preparation, and still another, the Manual on Standard Details and Methods of Detailing, already under study by Committee 315.

This type of committee work has been brought about by a recognition that practice generally has not kept pace with laboratory or analytical advancements. The literature pertaining to concrete design and construction is so vast that the engineer in normal practice cannot hope to segregate all of the significant developments and apply them to his own problems. By encouraging committee reports and papers of this sort, the Board hopes to help close the gap between technical development and engineering practice.

Impetus to committee activity in the field of standardization has come from a recent agreement with the American Society for Testing Materials, under which duplication of effort will be avoided in writing specifications in the field of concrete and concrete materials. Under this agreement, the A. S. T. M. will not undertake specifications and recommended practice in the field of concrete on matters of engineering design or construction practice, and the Institute in turn will not undertake the writing of specifications for "over-the-counter" engineering materials. This cooperative agreement brings to the Institute new opportunities and new responsibilities.

To fulfill its obligations under this agreement, the Institute must revise its procedure for the adoption of standards. At the present time the procedure as covered in the By-Laws is wholly inadequate for the new responsibilities. As a first step, the amendments to the By-Laws, submitted in the November-December JOURNAL, are proposed. These will be presented for action at this session. Under these amendments the regulations governing the adoption and revision of standards will be left for the Board to work out. This is a very desirable change, as by this means the regulations can be strengthened and improved as new situations develop without the necessary delays of By-Law changes.

The new procedure is now being thoroughly studied by the Board, and it is hoped that regulations will be established under which standards can be developed that are truly expressive of the best that science and the art can offer, and under which at the same time there will be no possibility of exploitation of special interests or unnecessary delays by obstructive tactics. It is intended also that these regulations will provide for periodic review and revision so that our standards will aid and not hinder the development of the art. In the measure to which the Institute's standards can achieve this ideal will they be accepted as the highest authority.

In conclusion let me urge on the members a feeling of responsibility for the continued progress of the Institute. It must be remembered

that the Institute is a mutual effort, and its contributions to the advancement of concrete practice must be made up from the contributions of its individual members. The Secretary's office and the Publications Committee do not produce information. Standards of recommended practice are not evolved from thin air. It is the individual experiences of our members wrought, through painstaking effort, into technical papers or committee reports that produce the Institute's contributions to the literature of concrete. You can help the Institute advance in its field of effort by contributing from your own experience and by bringing in new members and helping to fit them to some of the many tasks that remain undone.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MARCH-APRIL 1937

Concrete: Its Maintenance and Repair*

By R. B. Young†
MEMBER AMERICAN CONCRETE INSTITUTE

Repairing concrete is somewhat analogous to the treatment of disease. Before remedies can be correctly prescribed, the malady has to be diagnosed and before an accurate diagnosis is possible, the doctor has to have a thorough knowledge of disease, its various symptoms and treatments. The concrete doctor, in his field, needs similar knowledge if the treatments that he prescribes for ailing structures are to be successful, that is, he must know the causes for the deterioration of concrete and the symptoms of each.

There are many symptoms of deterioration that should be noted and evaluated in examining concrete; i. e., ravelling, where and how occurring; cracking, its position, extent, pattern and the presence of deposits therein; deposits of various kinds, both in and on the concrete, their location, kind, and color; the popping or cracking of aggregates; the color, texture, hardness, and ring of the concrete; the condition of the surface, whether it is absorptive, etched or scaled; the relation of deterioration to structural defects, to drainage, to joints, to the way the concrete was placed during construction and many similar conditions that require experience and a trained observation to interpret. Those interested will find information of this kind in papers previously presented to the Institute.

One of the difficulties in diagnosing the ailments of concrete is that they seldom occur singly. If a concrete is susceptible to attack, several agencies may combine to destroy it. This, of course, adds to the difficulty of diagnosis and in every case, an attempt should be made

†Testing Engineer, Hydro Electric Power Commission of Ontario.

*Presented at 33rd Annual Convention, American Concrete Institute, New York, February 23-26, 1937.

to determine which are the primary causes of the deterioration and which are secondary or only incidental.

When determining the significance of symptoms, consideration should be given to the age of the concrete and the rate of its deterioration. It is obvious that if concrete only a few years old, is weathering badly, it is more serious than if twenty years had been required to reach the same condition; thus in many cases, the necessity for repairs depends to a large extent on the rate at which the concrete is deteriorating.

As in medicine, so in repairing concrete, the diagnosis should be followed by the selection of suitable remedies and likewise there may be different treatments which will effect a cure. And as in medicine, there are also palliatives and there are cures, there are quack remedies and there are cases where our knowledge is insufficient to prescribe a sure corrective. But unlike medicine, where the patient is not likely to quarrel with the cost of the cure if the results are certain, the treatment of concrete has an economic side that must always be considered.

Maintenance and repair have much in common and in many cases it is difficult to say whether the remedial work being done is one or the other. In any case, the difference is largely one of degree and the methods are essentially the same. But maintenance, as generally understood, is work done during the ordinary course of operating a structure, while repair work involves major restorations requiring that, for prolonged periods, the structure be taken out of service, wholly or in part. In the discussion which follows, no attempt is made to separate the two, for the principles underlying both maintenance and repair work are the same and similar methods and materials are used.

There is no simple classification that can be followed in discussing the various methods used in the maintenance and repair of concrete. For convenience, we will consider them under the headings of surface treatments, waterproofing, pointing and caulking, patching, surfacing with mortar coatings and replacement. These overlap to a considerable extent and some methods may fall within two or more of the classifications, as for example, both caulking and surface treatments are used for waterproofing concrete structures.

By surface treatment is meant the application of some material to a surface as a means of protecting that surface or the body of the concrete behind it from the effects of weathering. These treatments may range all the way from painting to pneumatically applied mortar. They vary greatly in effectiveness and in their useful life.

PAINTING

Most painting of concrete is done for decorative purposes. This type of maintenance is not within the scope of the paper and those interested will find the subject discussed in the publications of the Institute.[1] Certain types, the asphalt, and cement paints are useful as surface waterproofings and they will be considered as such. The oil paints are handicapped by their liability to saponification. It is hopeless to apply an oil paint to a surface that is moist or likely later to become moist. Any moisture present in concrete rapidly becomes strongly caustic and very little, if in contact with an oil film, will cause it to saponify. Situations like this may sometimes be taken care of by the use of paints inert to these caustics, but if the paint films are themselves impermeable, they are likely to be pushed off by the moisture from behind. If they are permeable, efflorescence may form which, if it occurs behind the film will also push it off, and there are other difficulties. It is the author's experience, that as a class, paints have a very limited use in preventing the deterioration of concrete. Moreover, the best of paints have a relatively short life, and have to be renewed at frequent intervals to maintain their effectiveness as a protective coating.

Closely allied to the paints, are the surface waterproofings which are of many types. The most commonly met with are the transparent coatings usually consisting of various waxes or metallic soaps dissolved in a suitable solvent. Some are little more than clear varnishes. A few are compounds of iron which on application, are treated with sal ammoniac and a number are solutions or emulsions of asphalt. Some have merit, more have none and the best have to be used wisely for satisfactory results.

TRANSPARENT WATERPROOFINGS

The theory upon which the claims for the transparent waterproofings are based is sound but, as a class, they are very disappointing. They consist of a waterproofing material dissolved in a volatile solvent by which it is carried into the pores. On evaporation of the solvent, the waterproofing substance is left in the pores and so prevents the entrance of moisture.

Most of the transparent waterproofings, when newly applied, will waterproof concrete but few will continue to do so for more than a few months. Careful experiments planned to simulate natural weathering, were carried out by the Hydro Electric Power Commission of Ontario some years ago on a group of these transparent and colorless

(1) Painting on Concrete Surfaces—Report of Committee 407, JOURNAL, Amer. Concrete Inst., Sept. 1932, *Proceedings*, Vol. 29, p. 1.

waterproofing materials. Both mortar and concrete specimens were treated as directed by the manufacturers and tested for absorption at intervals up to two years. Parallel tests were run on untreated specimens. Many of the treatments had entirely lost their waterproofing properties in six months and none were of any value after two years. Earlier experiments by the Bureau of Standards gave much the same results,[2] while the more recent tests of Jumper, although carried out on a different basis, were equally negative.[3]

It is unfortunate that this is so, for in maintenance work there is a real need for a waterproofing material of this type that can be depended upon to give protection for a reasonable length of time, say from three to five years. The author keeps experimenting with these materials, always hoping that the next tested will be satisfactory but the experience of our British colleagues over a period of 75 years leaves little basis for this hope.[4]

The waterproofings of the varnish type are, for all practical purposes, paints with all their characteristics and shortcomings. Those that lay claim to deep penetration, approach the transparent waterproofings in their properties and usefulness. In the Commission's experiments, the varnish type of waterproofing has proven, as a class, the poorest of all.

IRON TREATMENTS

The iron treatments consist of very finely divided iron particles and sal ammoniac applied to a wet surface in the form of a grout. The iron is drawn into the pores of the concrete and changed there by the chemical into iron oxide. Since the iron, oxidizing, increases in volume, it is claimed that the pores of the concrete will be sealed. Jumper's tests would indicate otherwise.[3] The treatments are usually repeated one or more times and followed by a cement and sand wash.

Iron treatments have been known for many years and certain responsible organizations doing repair work use them regularly and, in the face of this, one would be foolish to condemn them. The author has had but limited experience in their use but in so far as they are applied solely as surface waterproofing treatments, his observations lead him to be very doubtful of their efficiency. On the other hand, he has seen some successful repairs in which they were used as a waterproofing and bonding medium under a patch or plaster coating and others where they were used as an admixture. Whether or not

---

(2) Technologic Paper No. 3, National Bureau of Standards, 1911.

(3) C. H. Jumper—Tests of Integral and Surface Waterproofings for Concrete—JOURNAL, Amer. Concrete Inst., Dec., 1931, *Proceedings* Vol. 28, p. 209.

(4) R. J. Schaffer—The Weathering of Natural Building Stones, Special Report No. 18, 1932. Department of Scientific and Industrial Research, Great Britain.

the repairs would have been equally successful without, is a moot question which circumstances make it impossible to answer, but as to the success of the repairs, there can be no doubt.

BITUMINOUS WATERPROOFINGS

The bituminous waterproofings differ from the paints in that the vehicle used evaporates entirely, leaving practically pure asphalt on the treated surface. They fall generally into one of two classes: the solutions or "cut-backs," in which asphalt is dissolved in a petroleum solvent; and the emulsions, in which water is used as a carrier. Asphalt has also been applied hot after being heated to bring it to the proper consistency.

Bituminous waterproofings, where used as paints, have the latter's shortcomings. They do not stand up well if exposed to direct sunlight but they can be protected from its effects by combining with them, asbestos fibre, sand, mica or other reinforcement; or, in the case of the asphalts, by coating them with some sun-resisting paint. They excel the average paint in their resistance to moisture for they are chemically inert to the various solutions ordinarily present in soil, atmosphere or concrete. They are weak mechanically and are unsightly but both of these features can be corrected in part where occasion requires.

At this point, it is well, perhaps, to enunciate a principle underlying the waterproofing of structures. Wherever practicable, the movement of water should be cut off at its source; for example, the logical place to waterproof a retaining wall is on the side next to the ground, a dam on its upstream face, a pipe or conduit from the inside. But this is not always possible and one of the most difficult repair problems is to cut off the flow of water effectively where it comes to the surface. In the discussion to follow, the different problems presented by the two cases should be kept in mind.

In considering surface treatments such as we have been discussing, it is evident that little could be expected of them if the surface is under pressure from behind but they might be very effective if they could be used between the surface and the source of moisture. It is because of their ability to maintain their effectiveness in contact with water that makes it possible to use bituminous waterproofings in this way and it is here that they offer their greatest usefulness.

The Hydro Electric Power Commission, in their maintenance work have, in several instances, used bituminous compounds to waterproof the inside of wheelpits where deterioration was being caused by the seepage of water through construction joints or porous concrete.

The treatments have not always been completely successful but in every case they have proved beneficial and reduced, if they have not entirely stopped, the passage of water. Some of the experience gained in this work may be of value to others.

Both emulsified and cut-back asphalts have been used but to date, the former have given the better results. They have an advantage over the cut-backs because they can be painted on a damp surface,—an important asset in hydro-electric work where it is not always possible to get a plant out of operation long enough to dry out the surfaces to be treated. A disadvantage of the cut-back asphalts is the fumes given off in drying, which in confined spaces proves a burden, if not a menace to the workmen. Whatever type is used, the surfaces to which it is applied, should be cleansed thoroughly and then primed with asphalt, well thinned to give maximum penetration into the pores of the concrete and so provide a mechanical anchorage for succeeding coats. This is followed by two or more coats until a film of sufficient thickness has been obtained; or better, a putty made of asphalt treated with cement or sand may be spread on with a trowel to the desired thickness, usually from $\frac{1}{16}$ to $\frac{1}{8}$ in.

BITUMINOUS MORTARS

Pure asphalts of the grades used in these treatments, are mechanically weak and if required to bridge cracks or cavities of any size, will punch through under pressure. With emulsified asphalt, this can be overcome by mixing with it portland cement in the form of a grout of creamy consistency. The amount of cement used varies with circumstances but the maximum for satisfactory results is from 10 to 12 lbs. per Imperial gallon. If greater mechanical strength is required, sand can be added to the mixture until the desired body is obtained.

The inclusion of cement in emulsified asphalt helps to solidify it by removing the emulsifying water. Where asphalt is used alone to fill a large crack or cavity, it is often found that the surface skins over, leaving the mositure trapped in the interior so that it never sets properly. The use of cement overcomes this trouble and at the same time, reinforces the coating.

The waterproofing of surfaces to be painted decoratively presents a difficult problem. Experiments by the Commission have failed to find any treatment, except cement plaster, that can be applied on the inside of a damp wall or one that shows dampness intermittently, which will definitely stop the movement of water from the back. The experiments indicated that sealing the surface with a coating chemically inert to the caustic solutions present in the concrete was not

sufficient, for even with evaporation cut off, the capillary movements of moisture continued and built up a pressure in the entrapped air. This pressure would either puncture the paint film with a myriad of minute holes, thus permitting movements of moisture as before, or, in the case of some of the asphalts, cause them to blister. The remarkable thing about these blisters was that the asphalt film was actually split, with one part remaining fast to the surface of the concrete, showing that the strength of the bond exceeded the tensile strength of the film. Iron treatments had little effect in sealing the surface, the only difference being that the paints did not fail so quickly as on untreated concrete. Some success was had using special paints with a rubber base, but for severe conditions the only remedy found entirely satisfactory was to apply a cement mortar plaster to the surface prior to painting.

Where conditions are favorable, a very satisfactory method of waterproofing concrete surfaces is by the use of a plaster coat of cement mortar. It cannot be applied in locations where there is active seepage, unless the seepage is collected and drained away. Well applied, it should withstand considerable pressure once it has set, so that frequently the drainage system put in during application can be plugged. The success of any plaster job depends on the preparation of the surface, the way the mortar is applied and its subsequent curing, and it is useless to put any but experienced men on this class of work.

The Commission has used cement mortar in this way with entire success. One job in particular was a very wet underground tunnel that had to be used as a passageway. It was waterproofed with cement mortar and after a period of curing was decorated with an oil paint. No difficulty from saponification was experienced.

A difficult waterproofing problem is to prevent moisture passing through cracks from the outside to the inside of a thin concrete, gunite or plastered wall. If the cracks are structural, of appreciable width and few in number, the most satisfactory way is to groove, caulk and point them. Where they are numerous, such as shrinkage cracks over the reinforcement in a thin wall, it is impracticable to treat them in this way. Here a coating of cement paint will frequently seal the cracks, or the same result can be had by using raw linseed oil. The linseed oil has the advantage that no curing or after treatment is required, as with the cement paint, but it will give a yellowish cast to the surface for a time. It will have to be repeated periodically, say every three to five years and it is only applicable to walls that are normally dry and not in contact with the ground or other continuous source of moisture. Where it can be used it is very effective.

It might seem that the use of linseed oil as recommended here, is inconsistent with the previous statements regarding transparent waterproofings. It is probable that some of the commercial waterproofings would be satisfactory if used in this way, but they are more costly. Why linseed oil has proved so satisfactory is a matter of speculation. In the author's opinion, it is because of its great penetrating power. It goes deep into the cracks, it contains nothing to be filtered out in its passage and once in the concrete, it either sets up and fills the crack or, if it should saponify, the soaps formed act as a seal.

Raw and not boiled linseed oil should be used for this work. Boiled oil is more viscous, does not penetrate as deeply and leaves an unsightly surface. Boiled oil is, however, frequently recommended as a sealing coat where the surface is later to be painted.

The passage of water through construction joints and structural cracks, especially if it is under pressure, is another problem of great difficulty. The logical place to stop a flow of water is at its source but it is not always practicable to unwater a structure to do this and even if it is, it still may be impossible to determine exactly where the flow originates. This is particularly true where there are intersecting vertical and horizontal joints, for one may feed the other, and the water may and often is travelling long distances before it appears at a surface. A further complication results where there are structural movements at the joint to be waterproofed.

CAULKING

Many methods have been tried in an attempt to solve this problem. Where the concrete is not subjected to freezing temperatures and the joints are narrow, seepage can be stopped at its exit from the concrete by caulking. In this case, the first step is to localize the seepage by the installation of a drainage system to collect and remove the water. Then the crack or joint should be channeled or chased and thoroughly cleaned. If the cracks are narrow and can be cut out, they may be caulked with oakum and sealed with either cement or bituminous mortar, but it is a waste of effort to use cement mortar for this purpose if there are structural movements in the concrete. This method is applicable where the seepage is under considerable head and, carefully done, it has been found satisfactory. Frequently, the first attempt at any one point will be unsuccessful, but a skilled and patient workman will ultimately get a job. (Fig. 1).

A variation of this method is to place a little plastic asphalt at the bottom of the channel before the oakum is driven in or to use two

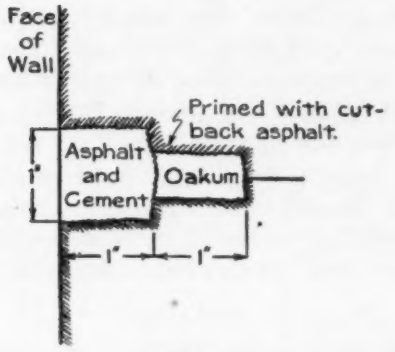


FIG. 1

layers of oakum with an asphalt ribbon between. The asphalt plastic should be used sparingly, especially if the pointing is done with a bituminous mortar, for there is a tendency, if the repair is liable to summer temperatures, for the oil in the asphalt to soften and bleed past the mortar at its contact with the concrete and so loosen the seal.

Proprietary compounds of rubber and asphalt have been used with great success but they are expensive and tricky. They cannot be melted, they must be applied hot and worked into the groove with tools. The surfaces to which they are to bond must be dry and, for best results, should be primed with a cut-back asphalt.

The same methods are applicable where the seepage is to be cut off at its source but, of course, no drainage is required. In many cases, the oakum may be omitted although its use will result in a more reliable job.

The only sure way of waterproofing seeping cracks or joints exposed to frost, is by cutting the water off at its source. Any drainage system that may be installed will freeze and put pressure on the caulking, and even where the caulking will stand the pressure, it will be forced out by the freezing of water trapped behind it, nor is it practicable to caulk at a depth sufficient to prevent such freezing.

Another reason why this type of repair is not always effective is that, where the seepage exists, there is a tendency for the moisture to spread into the adjacent concrete by capillary action giving a zone of saturated concrete that is susceptible to frost action. If trouble of this type has occurred, the caulking of the joint will not stop the capillary movements of moisture or the deterioration of the concrete.

A simpler problem is the waterproofing of cracks or joints to protect them against the effects of weathering. If there are no structural

movements to contend with, they can be grooved and filled with cement mortar, using methods described under patching to insure the bonding of the new concrete to the old and to prevent shrinkage of the mortar from the sides of the groove. Bituminous mortar, roofing plastics or emulsified asphalt may be used for the same purpose but in the case of the latter some difficulty may be experienced where it is subjected to low temperatures since it then becomes brittle. With any of the last three, it is necessary, for best results, to prime the surfaces of the prepared groove with a cut-back or thin emulsified asphalt.

GROUTING

The Commission has experimented with pressure grouting of horizontal joints in gravity walls but the results have not been satisfactory. In the first place, it was found that even with a very thin grout, the cement travelled only a short distance from the grout holes before it filtered out and the hole plugged. It might have been possible to have forced the cement farther, if higher pressures could have been used, but the latter would have involved the risk of lifting the section of wall above the joint. The method had other disadvantages. Even if a row of holes was grouted to form a watertight zone along the horizontal joint, the intersecting vertical joints, not being watertight, allowed seepage to reach the former back of where it had been sealed. An attempt was made to close the latter by drilling a vertical hole along the joint near the upstream face, which was to be filled with asphalt, but if there were any difference in the hardness of the concrete on either side of the joint, the hole could not be drilled straight and the attempt would have to be abandoned.

PATCHING AND PLASTERING

Patching, in the sense that it is used in this paper, refers to the replacement of small areas or volumes of concrete. Ignoring for the moment the preparation of the area to be treated, the success of any patch will depend largely on overcoming the tendency of the concrete to shrink after placement, and on securing a bond to the parent concrete.

Many methods of patching, claimed to overcome the twin evils of shrinkage and poor bond, have been put forward from time to time and almost every cement worker will have some way that he claims will absolutely assure success. In so far as the first is concerned, preshrinkage of the mortar has been found very effective. This is done by mixing it well ahead of use and letting it stand. The time required to obtain the necessary shrinkage varies with different cements, temperatures and mixes, and can best be determined by

experiment. Once shrunk, the mortar should be used without the further addition of water.

There are on the market materials, which when added to mortar or concrete, are claimed to counteract the effects of shrinkage. The author has examined several jobs in which they were used. With some, no cleavage between the mortar and the old concrete was found and the patch was tight; but with others using the same admixtures, it was observed that shrinkage had not been entirely eliminated and the patch had pulled away slightly from the parent concrete. As a whole, the results observed were no more satisfactory than those obtained by preshrinking the mortar.

There are several ways of insuring a good bond between the patch and the parent concrete. One frequently used, is to sprinkle dry cement on the wetted surface just before applying the patch material; another, is to throw the mortar forcibly onto the prewet concrete. Whatever the system used, the important thing is to get an intimate contact between the new and the old materials throughout the area to be patched.

The method of bonding most favored in the Commission's work, is to prepare the surface by brushing in successive coats of a thick cement grout. Each coat is allowed to dry until it becomes tacky to the touch before the next is applied. From three to five applications of grout are used, followed by the patch material well trowelled into place. This method is slow but it gets results and, where the color of the patch and the concrete are matched, patches can be made thereby that will almost defy detection.

If unsightly patches are to be avoided, it is necessary to match their color to the concrete. If the aggregates and the cement in the two are similar, this may easily be done by using the same proportion of each as was used originally. If different materials have to be used, some experimentation will be necessary to get the same color after the patch has dried out.

Finally, the patch should be thoroughly cured by keeping it covered with a damp cloth or otherwise wet for several days after being placed. Neglect of this simple precaution will defeat all the care taken in the preparation and workmanship. This cannot be emphasized too strongly.

Patching over large areas of shallow depth would be described more properly as plastering. In these cases the same methods apply as were described for patching. Surfacing to an average depth of approximately one inch and less has been done successfully, following the method of the Commission. However, unless skilled workmanskip is

used, the results are likely to be disappointing and the author does not advocate plastering for the restoration of surfaces, if it can be avoided.

An important factor in the success of patching is the preparation of the area or cavity to be restored. All loose and decayed material should be removed, the edges of the patch should be trimmed vertically to the surface or slightly undercut, and the surface of the concrete should be thoroughly cleaned of all dust and loose material by flushing with water. If compressed air is available, it may be used to advantage in addition to the water for cleaning the surface. The area to be treated should be well soaked a short time before applying the concrete but there should be no pools of standing water or excess moisture present when the patch material is placed and, while the surface should be wet, some of the natural suction of the old concrete should remain.

Where large areas or masses are to be restored or protected, the methods so far described are no longer adequate and the operation becomes essentially a major repair. Work of this kind requires not only the restoration and protection of the concrete and the removal of the causes for the deterioration but, frequently, the strengthening of the structure or the making of alterations thereto. It is not the purpose of this paper to consider the latter type of work, except to indicate that sometimes it is the governing factor in any decision as to methods to be adopted in making repairs.

Practically speaking, there is a choice of but two methods for the restoration and protection of large masses of concrete. The one, is to use reinforced mortar, pneumatically applied; and the other, ordinary concrete made, placed, and cured according to standard practice. The two may be used in combination on one job, but they remain distinctly different methods of treatment.

PNEUMATICALLY APPLIED MORTAR

Pneumatically applied mortar has been used for the repair of concrete for many years and on a variety of structures, and is a recognized method of repair. It has the advantage of being easily applied over large areas and to require but a limited amount of equipment. It has the disadvantage of having to be put on in thin layers and, if there is any depth to be built up, it requires many successive applications to complete the job. It has the further disadvantage that it has to be applied with great skill and its success is largely dependent on one workman, the nozzle operator. Lamprecht has described in some detail[6] the methods to follow and the precautions to be ob-

[6] J. Lamprecht—Concrete Restoration in Water Impounding Structures—JOURNAL, Amer. Concrete Inst., May-June, 1936; *Proceedings* Vo. 32, p. 532.

served in using pneumatically applied mortar, and they will not be repeated here.

The author has examined many structures where pneumatically applied mortar has been used. Almost without exception the mortar surfaces have been found cracked and, in many cases, carbonate deposits are present, indicating a movement of water through these cracks from behind. However, the direct passage of water through the mortar itself, is conspicuously absent. Where no reinforcement has been used, areas will frequently be observed in which the mortar has spalled from the concrete below or is loose, but there are a few jobs where it is intact and in good shape. Even where the mortar has been reinforced and dowelled according to the best practice and where the job has been well handled, areas of appreciable extent have been found separated from the concrete below.

Whether or not this lack of bond is due to poor workmanship or some other cause is not entirely clear from the author's observations. At first he was inclined to believe the former, but more recently he has felt that there was a strong possibility that in many cases, the mortar came away from the concrete after the work was completed, for far too many jobs, which had been done by skilled and responsible people and under competent supervision, had this fault. Recently, the Commission used both reinforced and unreinforced mortar applied pneumatically, on two structures and every square foot of each was inspected by tapping with a hammer and found properly bonded. These jobs will be reinspected periodically and if any breaking of the bond is discovered, there will be no question but that it occurred after the concrete went into service.

In the majority of structures examined, pneumatically applied mortar, in spite of its cracks and its occasional lack of bond, may be said to be serving its intended purpose reasonably well. While most of the structures inspected have been less than ten years old in these as yet its faults do not seem to have reduced its effectiveness to any great extent.

REPLACING WITH CAST CONCRETE

Ordinary concrete offers advantages in many types of repairs and hence is widely used for this purpose. It is particularly applicable where major restorations, additions or changes are to be made. Its use in these cases differs in no way from standard practice. As with any concrete construction, it should be made of sound materials, correctly proportioned, thoroughly mixed, and it should be handled and placed without segregation. Finally, it is most important that it be properly cured. These requirements and the reasons for them

have been discussed so often in the technical literature that they need not be repeated here.

The principal difference between the use of concrete in ordinary construction and in repair work is in the attention that has to be given to the preparation of the concrete to which the new is to be bonded. It is of the utmost importance that all decayed and defective material be removed from the sections to be restored, and that the surfaces be so prepared that the new concrete will knit with the old. In so far as bonding is concerned, the methods described under patching have been found satisfactory, but where it is necessary that the new concrete share the load, it should be reinforced and dowelled to the old to insure their acting as a unit.

PREPARATION FOR REPAIRS

It is not always easy to determine just how much of the deteriorated concrete needs to be cut away. In many cases, if an attempt were made to remove all material that was in any way weakened, little or none would be left; but where the old concrete is to be encased by the new, it is not necessary that the former should be removed until there is no further evidence of deterioration, provided that all loose material has been eliminated. Where the old and new concretes are to form a junction at a surface exposed to the weather or other adverse condition, it is important that the old concrete be perfectly sound or else it will be found that in a few years, deterioration will have taken place behind the repair. Many instances of this kind may be observed in examining reconditioned structures.

The proper preparation of the old concrete to receive the new is, of course, just as necessary with pneumatically placed mortar as with concrete; but with the former, the problem is more often that of preparing large, rather than local areas. Here again, it does not seem necessary to remove all deteriorated concrete provided the loose and decayed material is eliminated. But a good surface to which to bond the mortar is necessary and should be provided.

A method that has been used by the Commission in preparing plane surfaces of large area, is to chip with a narrow tool, shallow grooves three to four inches apart in both directions after the manner used by stone masons in trimming stone. A narrow chisel is better for the purpose than a broad one. The areas left between the grooves will usually spall as the second series of cuts is made, leaving a fresh surface for the application of the mortar or concrete.

ELIMINATING SHRINKAGE

One of the difficulties met with in casting in a new piece of concrete is to take care of shrinkage and prevent cracks forming between the

FIG. 2—GENERAL VIEW (SHORTLY AFTER COMPLETION) OF DAM WHICH WAS REPAIRED

old and new, especially along the top edge. If the volume of the piece to be cast is not too great so that it is possible to preshrink the concrete as described under patching, little difficulty will be experienced in this connection. If preshrinking is not practicable, the best way to provide against excessive shrinkage is to have the concrete as dry as it can be placed. This, combined with the proper preparation of the surface and a sufficient period of curing, will usually result in a satisfactory joint.

A method that can sometimes be used where shrinkage has to be absolutely eliminated is to dry-pack, using a mortar or concrete of a very low water content. For setting machinery or column bases, for underpinning or similar work, a 1:2 or 1:2½ mortar will be found satisfactory. It should be mixed with just sufficient water to hold it together when tightly squeezed in the hand. This mortar is placed by tamping or ramming vigorously and, of course, it should be thoroughly cured. Concrete can be similarly handled but the general practice of the Commission has been to use cast concrete to within three or four inches of the member to be supported and to dry-pack the space left after the former has sufficiently hardened.

TYPICAL JOBS

The methods outlined in this paper for the repair and maintenance of concrete have been used successfully on many jobs by the Hydro Electric Power Commission of Ontario and others, and before concluding, it may be of interest to describe briefly a few that are typical.

The first case is that of a large storage dam of the Ambursen type, approximately 1200 ft. long and 50 ft. high at its deepest point. Eighty-five buttresses of varying height, spaced on 15-ft. centers, carry a reinforced concrete slab ranging in thickness from 11 in. at the top to 28 in. at the bottom of the deepest section. (Fig. 2).

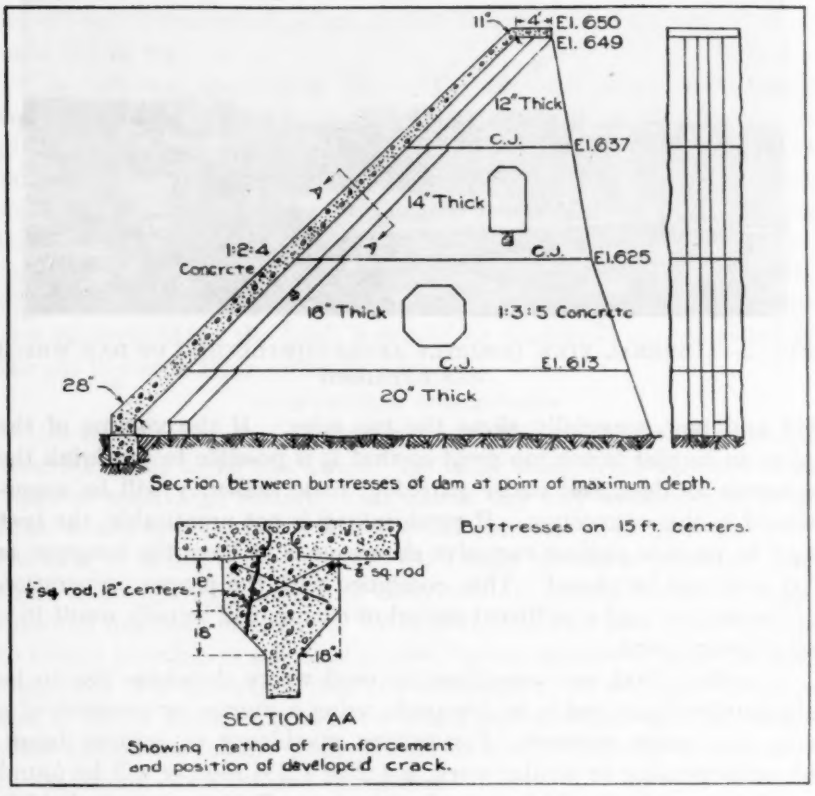


FIG. 3

The dam was built in 1914, of screened and washed sand and gravel of fair quality, proportioned 1:2:4 for the slabs and 1:3:6 for the buttresses. The workmanship and the quality of the concrete were good for structures built at that time.

Trouble with this structure was observed within ten years after completion and in 1927, an investigation was commenced to determine the cause. The concrete was watched closely for two years, by which time it was apparent that the deterioration was due principally to movements of water through cracks and porous sections, from front to back of the slabs and buttresses. The design of the reinforcement in the haunches of the latter contributed to this condition for no provision was made for tensile stresses in the direction of the supported slab. (Fig. 3). When the slabs did not slide freely on the haunches, as was intended, such stresses developed, and cracked a large number

FIG. 4—CRACK IN HAUNCH OF BUTTRESS CAUSED BY DRAGGING OF SLAB WHICH IT SUPPORTS

of the latter roughly parallel to the axis of the buttress piers (Fig. 4). This, in turn, permitted access of water to the concrete, which by pressure and capillary action, found its way to the surfaces. In passing through, the water rotted the concrete with the result that soft spots of considerable volume developed in 17 of the piers. On exploring these by drilling, it was found that the condition of the concrete became steadily worse, the closer one came to the front of the dam and it was evident that these spots were but the surface indication of a cancerous condition in those parts of the structure which were deteriorating.

Similar spots were found in a number of the slabs, principally in those parts in which the concrete had segregated during placing, but with a few exceptions, they were of small size. The disturbing feature about the deterioration of these slabs, was that the trouble was steadily

progressive and gave evidence of spreading beyond the segregated areas into sound concrete. There was also a tendency for the movements of water, which were the direct cause of the decay, to develop wherever a structural crack occurred in a slab, even in sound concrete, although in these locations, the deterioration was at a very much slower rate.

The water impounded by this dam was soft, but was not acid and ordinarily would not be considered corrosive to concrete. However, it has been the author's experience, that any water, even if hard, will, in time, rot concrete if that concrete is sufficiently porous to permit its movement from place to place.

When it became evident that repairs would soon be necessary, possible methods of restoring this dam were studied. Consideration was given to the construction of a new slab, to waterproofing the existing slab by various means, to converting the old dam to an earth dam and other schemes; but any of these would have been very costly and would have necessitated the shut-down of an important generating plant for a period of months, and the loss of valuable water. It was finally decided that the most satisfactory solution was to build a new dam of the same general type within the old, for this could be done piecemeal, only replacing as needed, those sections that had become weakened. The advantages of this scheme were that it did not necessitate the unwatering of the dam or the building of an expensive cofferdam, and only those piers or slabs that required strengthening need be done at any one time. It had the further advantage that it was not necessary to remove the decayed concrete from the old structure or even to waterproof it, as no dependence was being placed on the old dam for carrying the load.

Economically, this plan was attractive, for it spread the cost of the reconstruction over a long period. In fact, the money that had been set aside for renewals were more than sufficient to make the initial repairs and the present rate of deterioration is such that the costs of future reconstruction will be kept well within the limits of this amount so that at the end of the estimated life of the original structure, the then existing dam will be practically as good as new.

In the summer of 1929, one of the slabs developed a crack that was plainly due to a structural failure and immediate steps had to be taken to repair it. In as much as it was necessary to bring in considerable construction equipment for this work, it was decided at the same time to repair all of those buttresses that were badly decayed.

FIG. 5—METHOD OF REPAIRING BUTTRESS AND HAUNCH OF DAM

Note at left haunch trimmed back partly to allow for recess to hold slab; and at right, the system of reinforcing used. Dowels are carried through and into a similar section, added to the other side of buttress.

The general plan followed in repairing the dam was to provide new piers alongside the old, capable of carrying the full load of the structure. These were tied to the existing piers in such a way that the load would be transferred to them and they were provided with recesses at the front end into which a slab could be built when it became necessary to strengthen the old. Details of the methods are shown in the illustrations. (Fig. 5 and 6).

Screened sand and gravel similar in quality to that of the original construction were used in the repair work but so proportioned as to give a concrete having a 28-day compressive strength in excess of 3000 p.s.i. The concrete was cured by leaving the side forms in place for several days after casting and by keeping the surfaces wet. All joints with the old concrete into which water might penetrate, were primed with an asphalt solution and sealed with an emulsified asphalt putty.

This repair work has now been in service for seven years and in this time has proven satisfactory in every way. It is anticipated that further repairs to this dam will be required within the next two years and the same methods will be used but the amount of work to be

FIG. 6—COMPLETED REPAIRS TO HAUNCHES OF DAM

Note above, use of asphalt putty to seal joint; and below, replacement of a defective part of a slab.

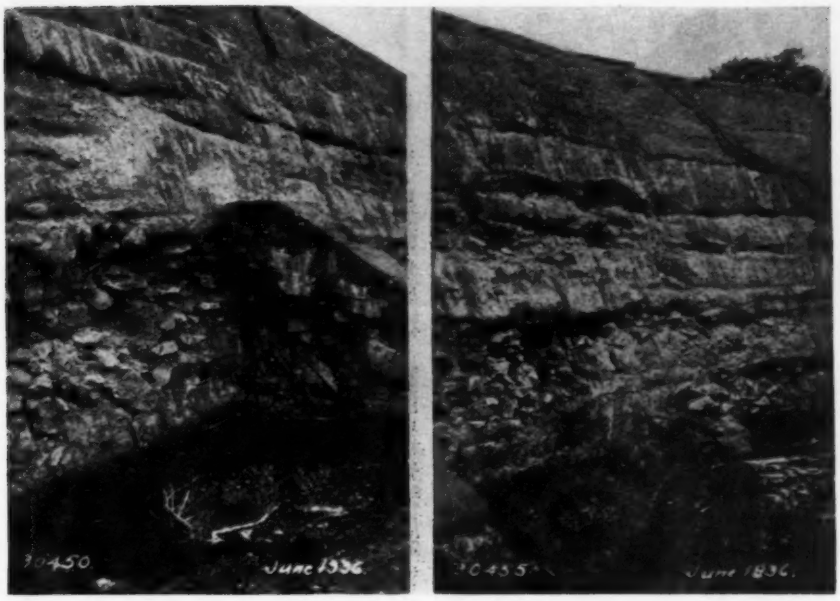


FIG. 7—TYPICAL SECTIONS OF DEFECTIVE GRAVITY WALL PRIOR TO REPAIR

done will be very much less than in 1929, for the rate of deterioration is slowing down, as more resistant concrete is reached. It is probable that before the dam has to be entirely rebuilt, the rate of deterioration on the original structure remaining, will have become so slow as to be negligible and that then further repairs will be unnecessary.

An interesting repair, which illustrates the use on one job of a number of methods, was the case of a small hydro-electric plant in Eastern Canada. Concrete has been used in this plant to build the canal and forebay walls, and for the powerhouse and superstructure foundations. The gravity wall forming one side of the forebay had been built of such a scant section that it was unstable. Part of this wall fell over shortly after the plant was first put into operation and the previous owners had thickened the deeper sections by adding concrete to the front and by the addition of a few small buttresses at the back.

About everything that could be wrong with a structure was wrong at this plant. The concrete was made from local pit-run gravel, and was probably dirty as used, for information obtained locally indicated that the pit from which it came had not been even stripped. No information could be obtained as to the proportions used but the

FIG. 8—INSIDE FACES OF WALLS BEFORE REPAIR

appearance of the concrete indicated very lean mixes. A great many plumbs were added in the forms and these were allowed to lie close to the surface and to each other.

The placing of the concrete was of the crudest, and the walls were everywhere a succession of fill planes, yet in spite of this, little segregation existed. The resulting concrete was porous and readily attacked by frost wherever seepage took place but except for its extreme porosity, it was remarkably hard.

Last spring, a section of the canal wall was shifted by ice pressure and it became necessary either to rehabilitate or abandon the plant. The former was decided upon and a study made of what should be done. Besides replacing the shifted canal wall, it was evident that the gravity walls had to be strengthened and made watertight, the wheelpits had to be waterproofed, and finally, it was advisable to protect the old walls from weathering.

Various schemes were considered and the method adopted was to build an 18-in. curtain wall of concrete which would blanket the entire upstream face of the gravity wall and thus effectively cut off any flow of water through it; this curtain wall would also help to strengthen the wall. A new toe was cast on the downstream side, extending from rock upward to a point where it would cover the worst of the disintegration. This was tied into the old concrete so that the two would act as a unit. A cap was built on top of the old wall and the new section at the front. The few feet of the old wall still exposed on the down stream side was then surfaced with two inches of gunite, reinforced with wire fabric, fastened to the old concrete with dowels.

FIG. 9—COMPLETED REPAIRS TO GRAVITY WALL. TOP SECTION IS PNEUMATICALLY APPLIED MORTAR; BOTTOM IS CONCRETE CAST IN PLACE

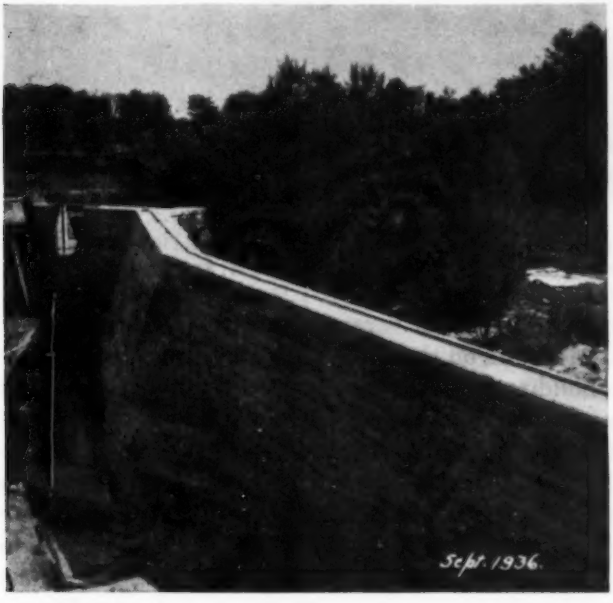


FIG. 10—VIEW OF FRONT OF GRAVITY WALL AFTER COMPLETION OF 18 IN. CONCRETE BLANKET WALL AND CAP

Details of the work and also the condition of the concrete prior to the reconstruction are shown. (Fig. 7 to 10).

The concrete used in the repair work was made from local sand and gravel, screened and of good quality, proportioned to give a dense and watertight concrete; its average 28-day strength was 3300 p.s.i. The concrete was pumped to the forms and was there consolidated by vibration. Vertical joints were treated with emulsified asphalt and horizontal joints were eliminated by casting all sections, rock to top, in one operation.

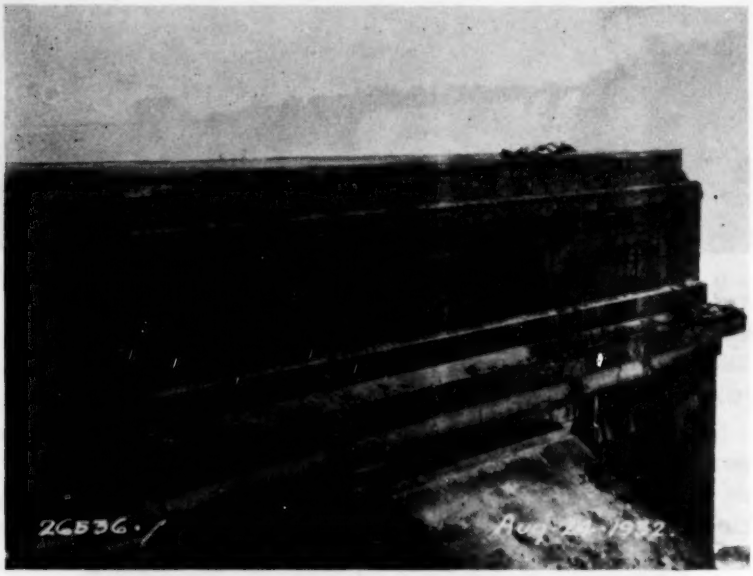


FIG. 11—FACE OF SECTION OF BUILDING 25 YEARS OLD. NOTE—WHILE CONCRETE LOOKS ROUGH, ACTUAL DETERIORATION IS SMALL

A third job, completed several years ago, illustrates the effect of rate of deterioration on the need for repair. The structure in question is a large building that has been added to from time to time. The first section was built about 1905 and the last in 1918. The building is exposed continually to spray and in cold weather to a great many cycles of freezing. In the course of time, parts of the building became deteriorated, but it was observed that the 1905 construction was in as good or better condition than that built thirteen years later. Considering the time the former had been in service, it was felt that the rate of deterioration was so slow that no repairs to this part of the structure were required. On the other hand, the poor condition of the

FIG. 12—CORNICE OF SECTION 12 YEARS OLD. NOTE ADVANCED DETERIORATION ON TOP STEP AND PATCH BELOW, ALREADY COMING AWAY

1918 concrete, indicated that early repairs of this section were inevitable, and it was decided to make these before the deterioration had proceeded so far that a great deal of rebuilding would be necessary. (Fig. 11 and 12).

The method of repair was very simple and effective. The trouble was mainly confined to the cornices and these were scaled of all loose concrete, built up to line and grade and covered with copper. The use of copper was decided upon because of the poor quality of the original concrete and the fact that the quantity that otherwise would have had to be removed entirely to eliminate decay, was so great that the copper covering was very little more expensive than complete restoration and, from the standpoint of permanence, offered a more certain repair. (Fig. 13 and 14).

Somewhat similarly, very successful repairs have been made to the upstream faces of piers which were unravelling at the water-line, by covering them with steel plates. These plates were anchored to the old concrete by bolts and by bending the back edges into chases cut in the sides of the piers. The space between the plate and the pier was filled with grout.

Some of these plates were removed after being in service a number of years and no disintegration of the concrete behind them was found. This may seem strange, but might be expected when it is remembered that the steel extends into water and the cycles of freezing and thaw-

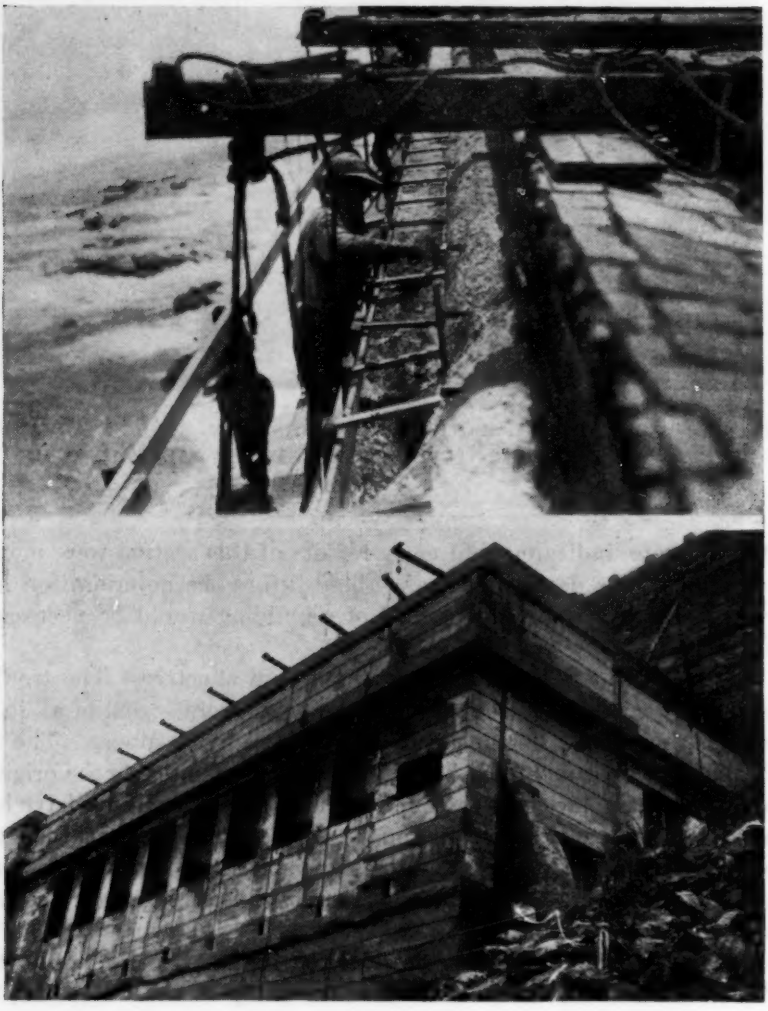


FIG. 13—METHOD OF BUILDING UP CORNICE TO TAKE COPPER. NO ATTEMPT MADE HERE TO REMOVE ALL AFFECTED CONCRETE, ONLY THAT WHICH WAS SOFT OR LOOSE

FIG. 14—COMPLETED REPAIRS TO CORNICE OF 12-YEAR OLD SECTION OF BUILDING

FIG. 15—USE OF STEEL PLATES TO PROTECT POROUS CONCRETE IN NOSE OF SLUICE PIER

ing which the concrete behind the plate would receive annually, would be reduced thereby to a very small number compared to similar concrete unprotected. This method offers a very satisfactory means of repairing the fronts of piers without the necessity of taking the structure out of service. (Fig. 15).

In repair and maintenance, the policy should be that of the Mikado of Gilbert and Sullivan fame, which was "To make the punishment fit the crime." There is no one method that is best for all conditions and any engineer charged with the responsibility of restoring a concrete structure, should approach the problem with an unbiased mind. He should first determine the causes of the difficulty, devise means for their elimination and then put the structure back into satisfactory operating condition, using whatever methods offer the required results with a certainty of permanence, with an appearance in keeping with the prominence and purpose of the structure, and with due regard to economy.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by June 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MARCH-APRIL 1937

Some Comparisons of European and American Concrete Practice*

By F. R. McMillan†
MEMBER AMERICAN CONCRETE INSTITUTE

It is impossible in a single paper to present even a general summary of the observations during a ten-week period devoted to the single purpose of studying concrete construction in Europe. This paper, therefore, will be limited to the presentation of a few examples of European concrete structures sufficiently diverse and typical to afford some comparison with American practice.

In France, the work is characterized by boldness of design and execution, and extreme economy of materials.

Fig. 1—A 3-hinged arch railroad bridge, built in 1907-1908 by Harel de la Noe. The latticed arch ribs and the slender columns required a minimum of materials. The end piers are a combination of concrete and stone masonry, typical of the work of this engineer. This section of France (Brittany) contains many examples of his designs in which he combined stone, brick, concrete, and cast iron.

Fig. 2—Vezins Dam, built 1929-1931. Multiple arch structure with exceptionally thin members, maximum height 108 ft. Arches 6.7 in. thick at the top and 25.6 in. at the base of the deepest arch. The up-stream face was covered with gunite and painted with a bituminous preparation. Forty thousand barrels of portland cement were used in the superstructure (7.1 U. S. sacks per cu. yd.). Twelve thousand barrels slag cement were used in the foundations and heavy sections (6.3 sacks per cu. yd.).

*Presented 33rd Annual Convention, American Concrete Institute, New York, Feb. 23-26, 1937.
†Director of Research, Portland Cement Association, Chicago, Ill.

FIG. 1—SEE TEXT, PAGE 395

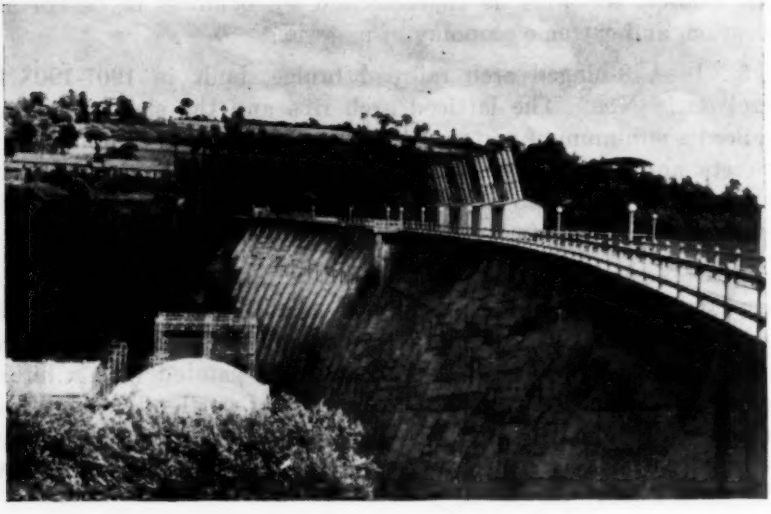


FIG. 2—SEE TEXT, PAGE 395

FIG. 3

Fig. 3—General view of the Plougastel Arch near Brest, that masterpiece of a master craftsman, E. Freyssinet, completed in 1929. The construction was described by Mr. Freyssinet in an A. C. I. paper (Proceedings, v. 25, 1929). High alumina cement was used up to the high waterline, high strength portland cement above. Concrete mixture 7.2 sacks per cu. yd. throughout. Spans are 612 ft.; the arch ribs are hollow.

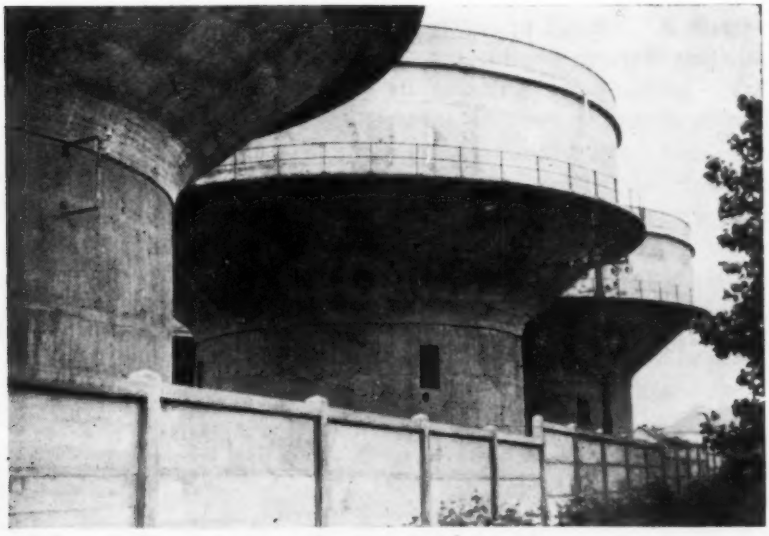


FIG. 4

Fig. 4—Reinforced concrete water tanks for the city water supply at Angers, built in 1932. Workmanship generally good.

Fig. 5—Making a stack serve the extra purpose of supporting the water tank at the cement plant of the Lafarge Co. at Angouleme, built in 1931.

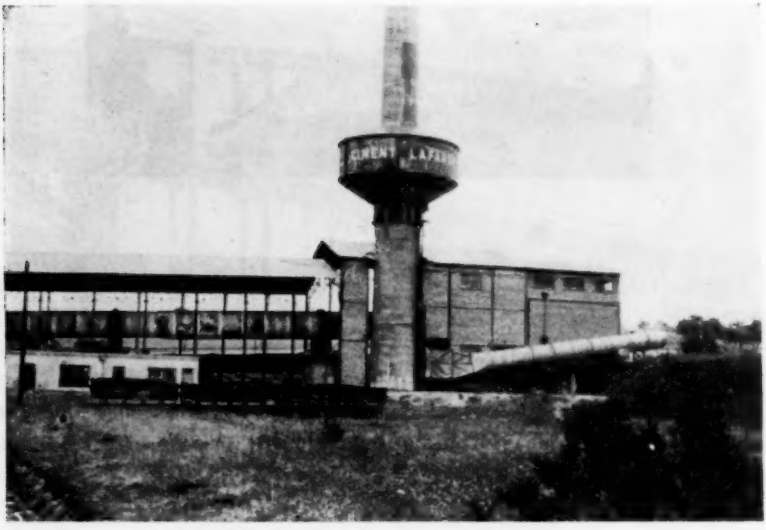


FIG. 5—SEE TEXT, PAGE 397

FIG. 6

Fig. 6—A bowstring truss bridge about 20 miles west of Nice. Structures of this general type are to be seen almost everywhere in France. The hand rails in this structure were extremely slender and are now spalling badly.

FIG. 7

Fig. 7—One of the most interesting sights in France. A housing project under construction in which precast units of concrete and steel frame are brought together into an amazing combination. This picture shows a unique fire escape on a 16-story unit. A closeup view is shown at the right and a distant view at the left. The series of round shafts which support the steps, and enclose the stairway, are of precast concrete. The treads are thin, reinforced concrete slabs fitted over the shafts.

Fig. 8—Chambon Dam, completed in 1935. It contains 410,000 cu. yd. of portland cement concrete. The mixture varied from 4.3 sacks on the upstream face to a minimum of 2.6 sacks per cu. yd. in the interior. On the downstream face, the mixture contained 3.9 sacks. Plum stones were used in the interior.

Fig. 9—Waggital Dam in Switzerland, built in 1923-24 of portland cement concrete. The original downstream face of concrete was covered with stone in 1930 because of surface disintegration. The altitude here is about 3000 ft., temperatures occasionally go as low as —20°F. Three types of concrete used: 3.3, 3.9, and 5.2 sacks per cu. yd., respectively. The richer concrete was used on the upstream face and in the entire upper 50 ft. of the dam. The leaner mix was used in the body of the dam, increasing the cement content towards points of maximum stress. The average in the downstream face was considerably under 4 sacks per cu. yd.

Fig. 10—Langwies Viaduct in Switzerland, built 1912-1914. The arch is a 328-ft. span, 220 ft. above the gorge. The workmanship

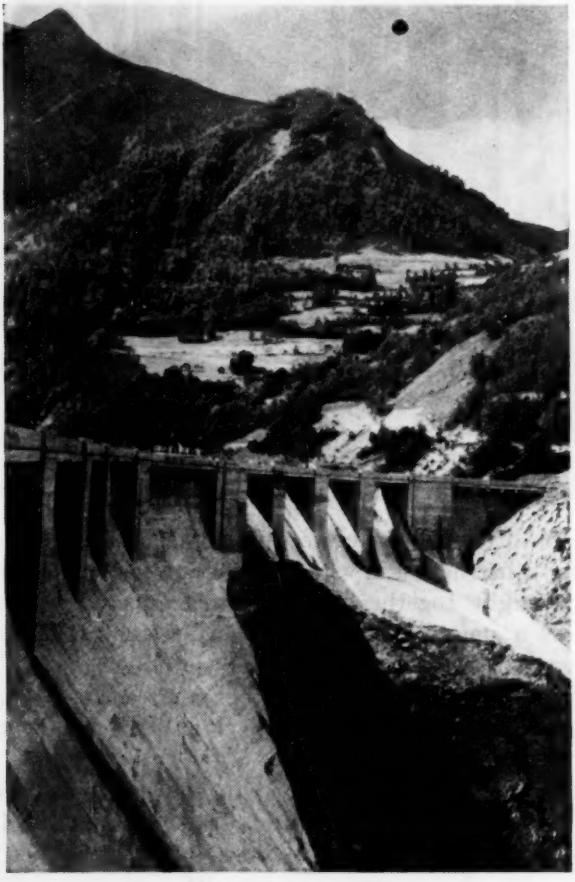


FIG. 8—SEE TEXT, PAGE 399

here was generally good. There was evidence of some patching at the time of construction, but no spalling or disintegration was found. The elevation here is about 4500 ft.

In Germany, the structures are definitely more simple in design and generally more massive. Most interesting of the observations here were in connection with the construction of the Reichsautobahn, the national system of automobile roads, being built as an unemployment relief measure. When completed this will be a system of super highways probably unequalled anywhere.

The roads are all double driveways, separated by a 15-ft. strip. Each driveway is of concrete, 24½ ft. wide, with a 3-ft. strip of asphalt

FIG. 9—SEE TEXT, PAGE 399

FIG. 10—SEE TEXT, PAGE 399

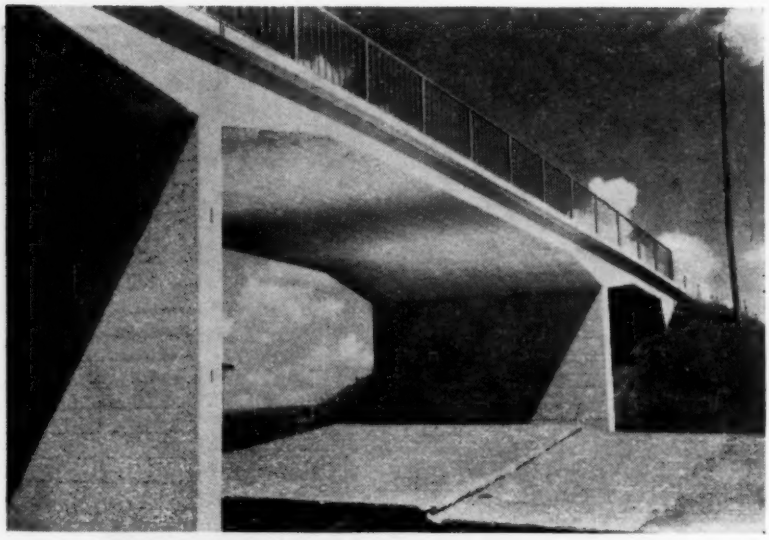


FIG. 11

on the outside and a 15-in. strip on the inside, making a total of approximately 29 ft. for two driving lanes.

The roads do not follow old locations. They were laid out with a single purpose of providing rapid transportation between principal centers. There are no grade crossings in the entire system, and the roads can be entered only at widely spaced points in the vicinity of important population centers. They were designed to accommodate buses traveling at speeds of 70 miles per hour.

Fig. 11—An overhead crossing of simple detail. Two spans, 39 ft. each, with overhanging cantilevers at ends. The bridge is symmetrical about the pier in the foreground. The canvas-covered frames prominent in the lower part of the picture protect the freshly placed concrete of the road slab. The workmanship on all of these structures appeared to be rather good. The bush-hammering gives a very pleasing appearance that could be copied to good advantage in any country.

Fig. 12—An overhead bridge of massive construction. The piers and sides of bridge bush-hammered.

A structure of particular interest was the Elbe River crossing of the Reichsautobahn (no picture shown). The point of interest here was the use of five types of cement in the piers supporting the 110-ft. long approach spans. As completed each pier had five types of concrete, each made with a different cement as follows: (1) A concrete seal at the bottom of the caissons made with high alumina cement,

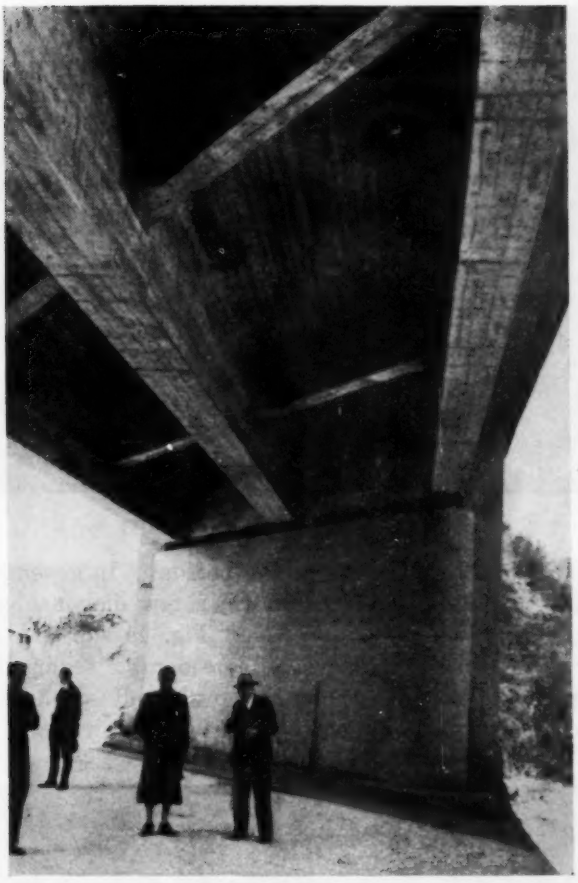


FIG. 12—SEE TEXT, PAGE 402

250 kg. cement per cu. meter; (2) from the seal to the surface of the ground, trass cement, 275 kg. per cu. meter; trass and clinker proportions 30-70; (3) body of the pier, portland cement, 325 kg. per cu. meter; (4) Upper 5 ft. of pier, high early strength cement, 360 kg. per cu. meter; (5) Pedestals with a super high early strength cement, 360 kg. per cu. meter. The cement quantities represent a range from 4½ to 6½ sacks per cu. yd.

Fig. 13—Little Bleiloch Dam, in central Germany, built 1930-1931. The cement used was a mixture of portland cement and a finely ground admixture of granulated blast-furnace slag called Turament in the proportions: 1 of cement to 2 of admixture. Three grades of concrete were used, varying in richness from 4.5 sacks per cu. yd. at

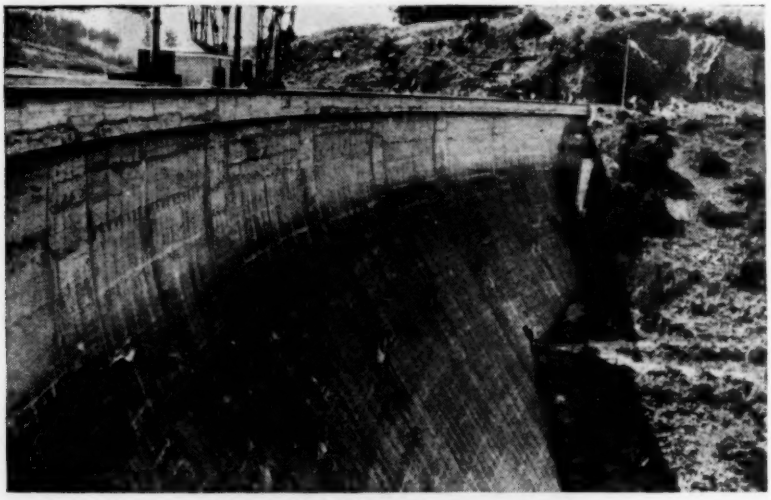


FIG. 13—SEE TEXT, PAGE 403

the top of the dam to 6.3 sacks at the bottom. In general, the dam proper is in good condition. There are a few diagonal cracks, not prominent, showing only slight seepage. Fig. 14 shows a horizontal slab appurtenant to this dam, where considerable scaling has developed. Inspection tunnels of the dam showed only small leakage and carbonate deposits.

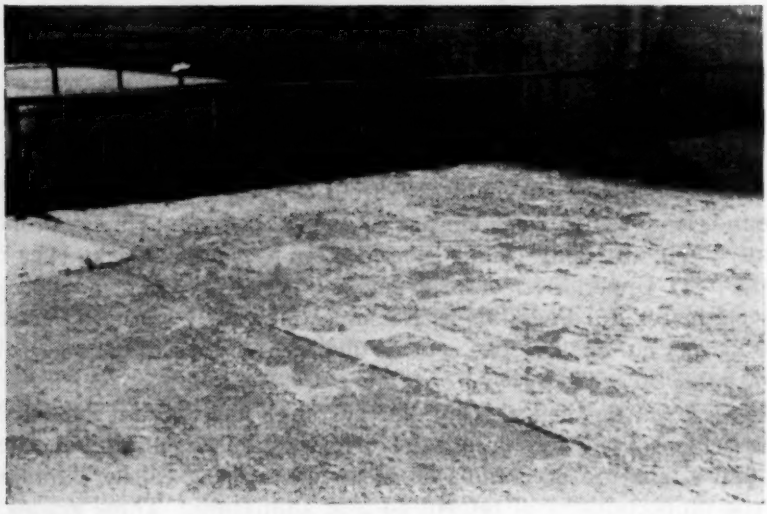


FIG. 14

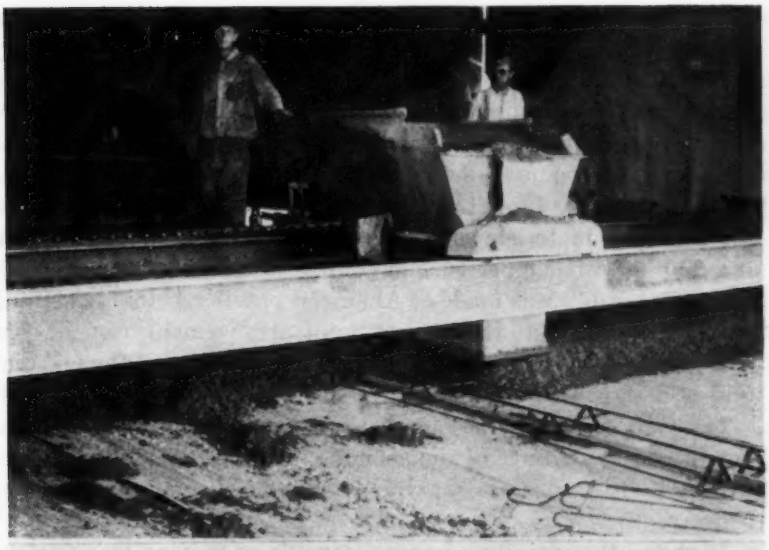


FIG. 15—(ABOVE) AND 16—(BELOW)

ROAD CONSTRUCTION AND MACHINE METHODS

The road construction on the Reichsautobahn was extremely interesting. We were able to inspect this at a number of different points, at no two of which were the methods exactly the same. Most of the work was being carried out with extensive machinery setups.

The few pictures will show some of the more interesting features of the construction operations. The procedure has not been entirely standardized, either as to construction methods or reinforcing details. There is a leaning towards the one-course construction except where aggregate of good quality for the wearing surface is not at hand. At these places a base course is laid with a local aggregate and materials of desired quality are brought in for the wearing surface. The mixes are suited to local materials with the cement content ranging between 300 and 350 kg. cement per cu. meter (5.3 to 6.2 sacks per cu. yd.).

An extensive study was made of American practice before beginning this program and the slab thicknesses, joint arrangement, cement content, etc. are based largely on American experience. All of the machinery used is designed to travel on supports spanning the entire roadway. The rails on which these travel are firmly supported on strips of concrete laid in advance of the general construction, and which are later covered with asphalt to provide a dark marker at each edge of the roadway.

Fig. 15 shows the spreading hopper, capable of moving transversely on the supporting frame or longitudinally with it. Note the accurate level to which the concrete is spread by this machine. The mix is both dry and harsh, aggregate up to 2-in. maximum. Note the tent protecting the concrete from rain or sun. This is carried on separate rails about 3 ft. outside of those supporting other construction equipment.

Fig. 16 shows the surface after the first pass of the finisher. This machine has three screeds; No. 1 and No. 3 move across the slab, with a range of travel of about 3 in., No. 2 screed is vibrated in a vertical direction at 2000-3000 r.p.m. Several passes were required to finish completely the type of concrete being placed at this point. The vibration was not used in all passes. Transverse joints are placed about 50 ft. apart with a dummy joint in between. The center joint is also a dummy. In some cases a sawcut is made above the separating board at dummy joints after the concrete has hardened.

Fig. 17 shows one of the finished roadways with the reed mats in place for curing. These are kept wet for 28 days. In the foreground of this picture is the narrow strip of concrete marking the inner edge of this driveway. On the opposite side, the similar strip is one meter wide, mentioned above.

Fig. 18 shows placing operations on a strip of road in which minimum of machinery and a maximum of hand labor are being used. Before starting this operation an experimental short section was made which indicated that the road could be built at about the same

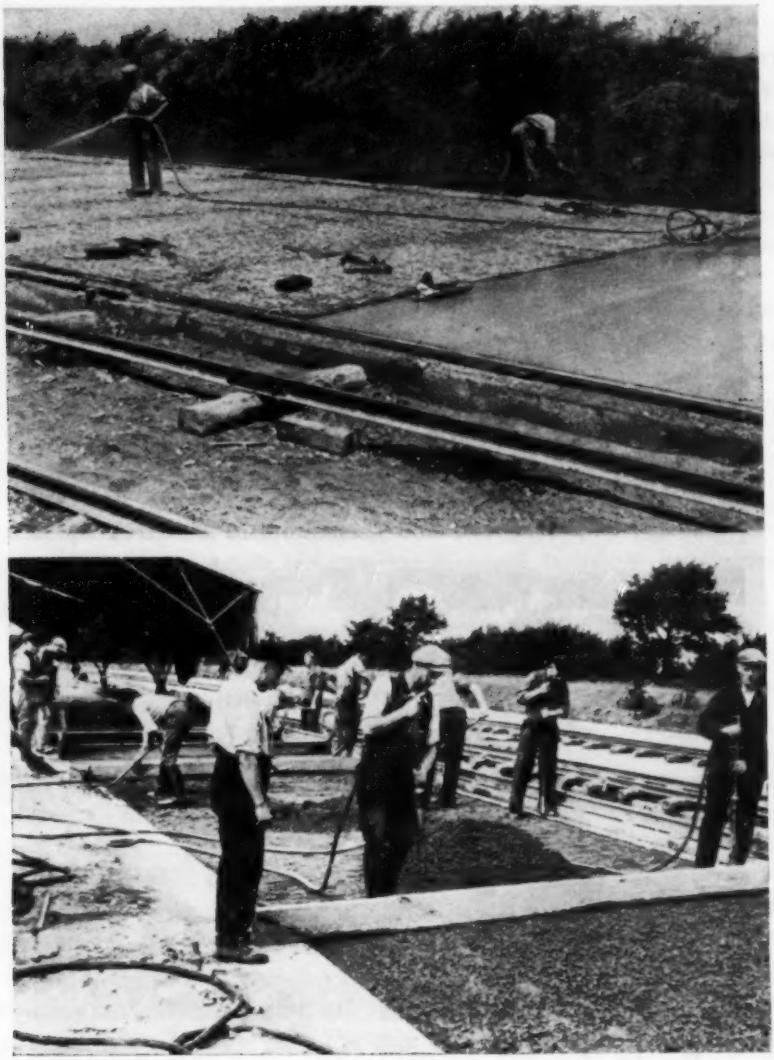


FIG. 17—(ABOVE) AND 18 (BELOW)—SEE TEXT, PAGE 406

cost and provide work for a greater number of men than is possible with machine placing. The slab is placed in four uniform layers, each about 2 in. thick, and tamped with the light, air operated vibrators, seen in this picture. These give about 420 strokes per minute and permit a very rapid compacting of the surface.

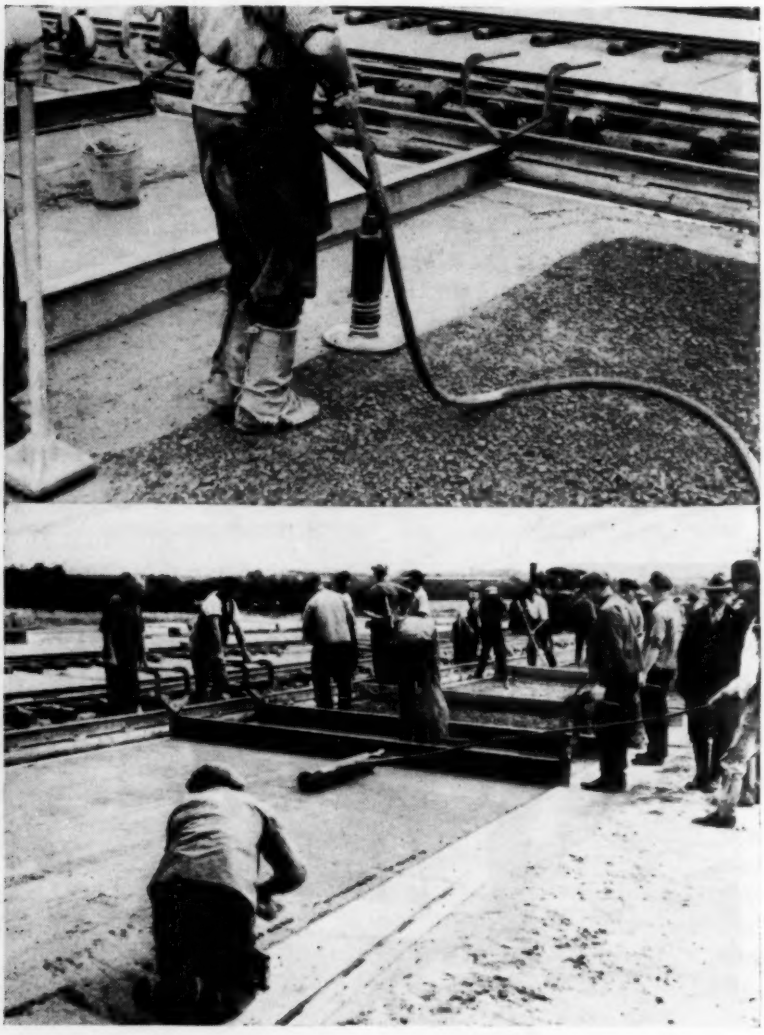


FIG. 19—(ABOVE) AND 20 (BELOW)—SEE TEXT, PAGE 409

Only half of the driveway is placed at a time by this method. The operations are rather clearly seen in the pictures. In the immediate foreground of Fig. 18, the concrete for the third layer has been spread and screeded, waiting for the tampers. Beyond the strike board, the tamping of the third layer is being completed, and part of the material for the fourth layer has been deposited. The first and second layers

are placed by this same procedure. Welded mesh reinforcement is placed between the second and third layers.

A somewhat different procedure is followed in compacting the fourth layer. This is finished with a rapidly vibrating tool (1700 r.p.m.) shown in Fig. 19. This is moved over the surface, thoroughly compacting the concrete in the thin layer. Some hand tamping is also done in the corners and along edges. The final finishing is done in several stages. Just beyond the power tamper in Fig. 19 will be seen an I-beam which is used for the first operation of the final tamping, and beyond this a second one for the final tamping.

Fig. 20 shows the same operations viewed from the opposite direction. In the foreground is the final finishing with the hand float; just beyond that a roller, and then the two hand tampers. Cores taken from experimental sections built in this manner showed no separation between layers and somewhat higher strengths than cores for similar mixes placed by machinery.

Too many exceptions and qualifications are necessary to permit, in the space available, any sweeping characterization of concrete practice in Europe. Certain outstanding features, however, justify some closing remarks.

On the Reichsautobahn in Germany the road construction is the equal of the best at home both as to methods employed and the results obtained. The bridge work on this system appears to be fully on a par with American practice.

For the older bridge structures in Germany, Switzerland, France, and England, the general appearance of the concrete is much the same as in the United States. The exposure, however, is definitely less severe than in most of the U. S., which indicates that our materials and workmanship may be somewhat better. For boldness in design or daring execution we have nothing to compare with some of the structures in France.

In dam construction the Europeans have gone to extremes in economy of materials that are not even approached by Americans. In massive work they vary the mix throughout the structure, using the minimum of cement content which will meet the requirements of strength or water pressure. Slag cements and blends with other materials are commonly though not exclusively used in mass construction. These economies would hardly be justified in the more severe exposure of our climate.

The workmanship on these dams is not the equal of the best at home, such as those being built by the different departments of our Federal government, but on the other hand we did not see in Europe

any of those "horrible examples" of bad construction which in too many cases are a blot on American practice.

It would be ungrateful to close without acknowledging the courtesies extended us by the European engineers. Everywhere we were greeted with friendliness and cordiality. Inspections were arranged, appointments made, and itineraries planned, sometimes at considerable personal sacrifice, but always with the one object of making our trip pleasant and profitable.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by June 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MARCH-APRIL 1937

The Effect of Vibration on the Strength and Uniformity of Pavement Concrete*

By F. H. Jackson†
MEMBER AMERICAN CONCRETE INSTITUTE
and W. F. Kellermann‡

INTRODUCTION

The possibility of using high frequency vibration as an aid to placing pavement concrete was first investigated by the Bureau of Public Roads in 1931.[1]. The equipment used consisted of a standard double screed finishing machine, with two electric vibrators mounted on the front screed and one on the rear screed. For the purpose of demonstrating its possibilities an experimental concrete pavement was constructed at Arlington, Va. It consisted of a series of sections, 9 x 9 ft., with means provided for securing large test slabs from each section. It was felt that the true effect of this method of placing and finishing could only be determined by the construction of full-size sections, using regulation contractors' equipment. These tests indicated sufficient improvement in quality to warrant further studies in which improved types of vibratory equipment could be utilized.

SCOPE OF THE WORK

In the present report the results of these further studies, which included tests on 250 sections, each 8 ft. long by 10 ft. wide, are presented and discussed. Except for a few 10-in. slabs cast for comparative purposes, all of the sections were 7 in. deep. The width was changed to 10 ft. to permit taking five flexure test slabs each 24in. wide from each section. Two coarse aggregates, Potomac River

*Presented 33rd Annual Convention American Concrete Institute, Feb. 23-26, 1937. A complete report of this investigation, including all detail test data upon which the discussion in this report is based, appears in *Public Roads*, Vol. 18, No. 2, *April*, 1937. Copies may be obtained from Superintendent of Documents, Washington, D. C., for 10 cents each.

†Senior Engineer of Tests, U. S. Bureau of Public Roads, Washington, D. C.

‡Assoc. Materials Engineer, U. S. Bureau of Public Roads, Washington, D. C.

[1]*Public Roads*, Vol. 14, No. 8, Oct., 1933.

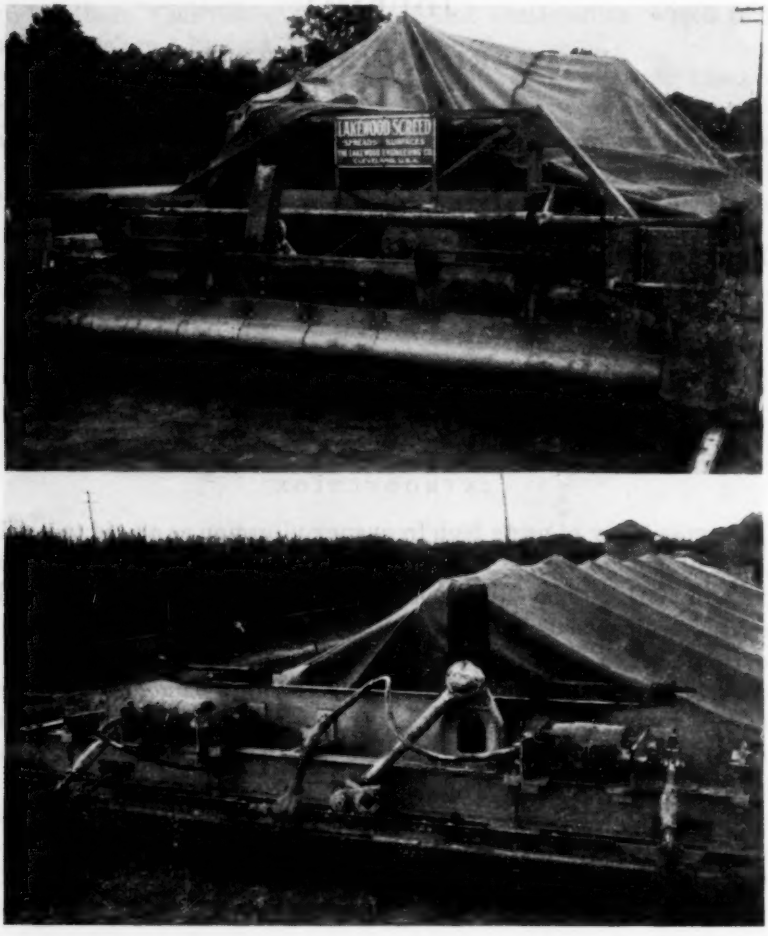


FIG. 1—MACHINE A, SHOWING BULL-NOSE FRONT SCREED
FIG. 2—FRONT SCREED OF MACHINE A, REAR VIEW SHOWING MOUNTING OF VIBRATORS

gravel and a crushed limestone from Riverton, Va., were used. Two types of vibrating equipment, including vibratory screeds and the vibrating pan mounted independently of the screeds, were investigated, using paving mixtures varying considerably in harshness and consistency. The construction and test procedures followed, in general, those previously employed and will not be described in detail in this report.

TABLE 1—PROPERTIES OF AGGREGATES

(a) Physical Properties.

	Fine Aggregate	Coarse Aggregate	
		Gravel	Crushed Stone
Bulk specific gravity	2.56	2.57	2.76
Absorption, per cent	1.40	1.14	0.32
Weight per cu. ft. (dry-rodded), lb.	110	109	103
Voids, per cent	31	32	40
Wear (Deval) per cent	——	8.4	3.4

(b) Sieve Analyses.

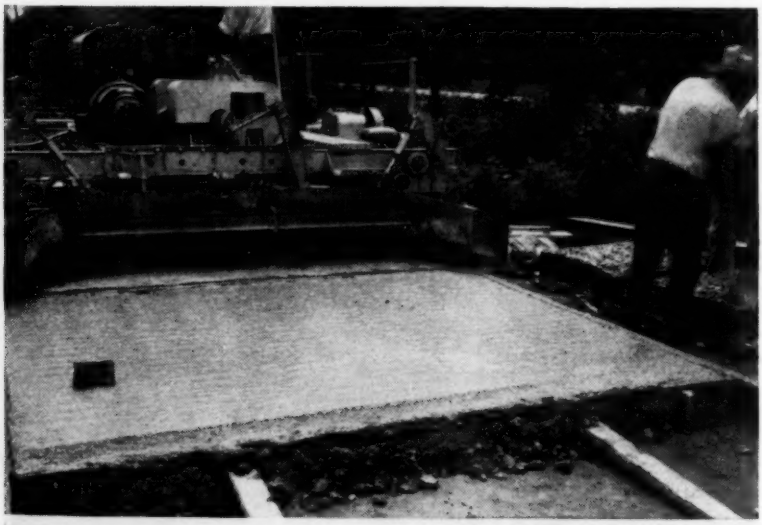
Retained on 1½-in. sieve, per cent	—	25	18
" " ¾-in. " "	—	64	71
" " ⅜-in. " "	—	87	92
" " No. 4 " "	5	98	99
" " No. 8 " "	22	100	100
" " No. 14 " "	34	100	100
" " No. 28 " "	51	100	100
" " No. 48 " "	84	100	100
" " No. 100 " "	97	100	100
Fineness modulus	2.93	7.74	7.80

MATERIALS

A single lot of portland cement meeting the usual requirements was used throughout the work. The fine aggregate was Potomac River sand having a fineness modulus of approximately 3.0. The coarse aggregates were used in approximately equal amounts and were handled in two separated sizes, 2 in. to 1 in. and 1 in. to No. 4. The two sizes were combined in each batch in the proportion of 60 per cent of the larger to 40 per cent of the smaller size. The grading and physical characteristics of the aggregates are shown in Table 1.

TYPES AND OPERATION OF VIBRATORS

Machine A, used on 144 sections, was of the type employed in the 1931 tests except that it was equipped with a heavier and wider front screed. (Figs. 1 and 2.) Power for operating the screeds and for driving the machine was derived from a gasoline engine mounted on the finisher. Two electric vibrators were mounted on the front screed and one on the rear screed. They were driven at a speed of 4,000 r.p.m. by a portable gas-electric generator set mounted on the finisher. The normal forward speed of the machine was 8 ft. per min. Except for the fact that the screeds were vibrated, this machine was



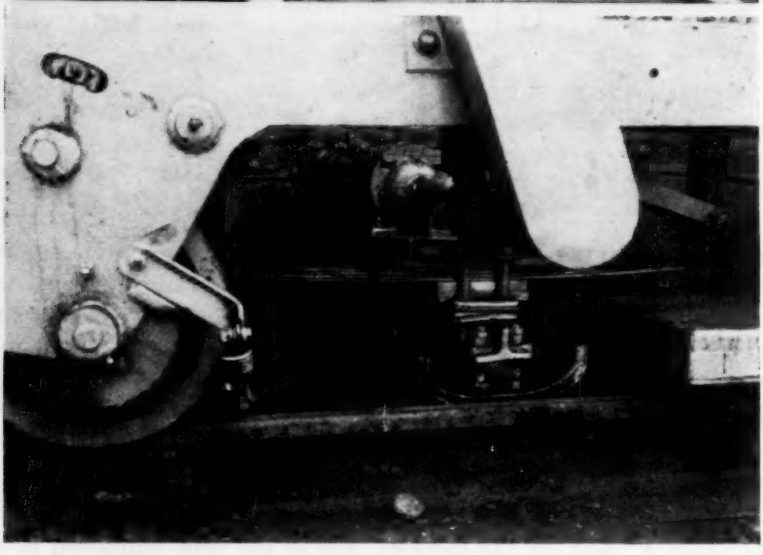


FIG. 3—FRONT VIEW OF MACHINE B

FIG. 4—MACHINE B, SIDE VIEW, SHOWING MOUNTING OF VIBRATING PAN

operated in most cases in the usual way; that is, two complete passes over the concrete.

Machine B was used on 106 sections. It is an all-electric mechanical finishing machine equipped with two screeds and a vibrating pan mounted between them. (Fig. 3 and 4.) The vibrating pan was

shaped like a trough and made contact with the concrete for practically the full width of the pavement. It was adjustable so that it could be raised clear of the concrete or could be lowered into the concrete a predetermined depth. It was equipped with two electric vibrators of adjustable amplitude operating at 3,600 r.p.m. There were no vibrators on the screeds. Power for running the machine, operating the screeds and running the vibrators was supplied from a gas-electric unit mounted on the finisher.

In the standard operation of machine B the vibrator was operated during the backward pass of the finisher as well as during the forward pass. By doing this a ridge of mortar was left at the beginning of the section which materially aided in the finishing operations. The height above the forms to which the concrete was struck varied with the consistency and harshness of the mix. The objective sought was to have sufficient surplus concrete so that, after vibrating, the surface would be at grade. For the second pass the machine was operated in the usual way without vibration.

PROPORTIONS

As in the previous tests, the effect of vibration was measured by comparing the properties of the vibrated concrete with those of a standard finished paving mix having a cement content of 6 sacks per cu. yd. and a slump of about 2½ in. This will be referred to as standard or "base" mix. For the first 42 sections (Series A), the proportions of the base mix were designed to give a workability factor (b/b_o) of approximately 0.76 (dry and rodded) for both gravel and stone. Corresponding values for the sand-cement ratios (a/c) by absolute volume were 2.10 for gravel and 2.53 for stone. This mix proved somewhat harsh for standard finishing, necessitating a slight increase in the amount of sand. The revised proportions for the base mix as used in Series B called for a workability factor of 0.73, with sand-cement ratios of 2.24 for gravel and 2.67 for stone.

In designing variations of the base mix for vibration the possibility of effecting economies through cement saving was considered as well as that of improving quality. To obtain the comparisons desired, the base mix was varied in three different ways. In one, coarse aggregate only was added to the base mix, the water-cement ratio and the sand-cement ratio remaining constant. This resulted in reducing the cement content and slump in proportion to the amount of coarse aggregate added. In another, both fine and coarse aggregates were increased, the water-cement ratio still remaining constant and the cement content and slump being still further reduced.

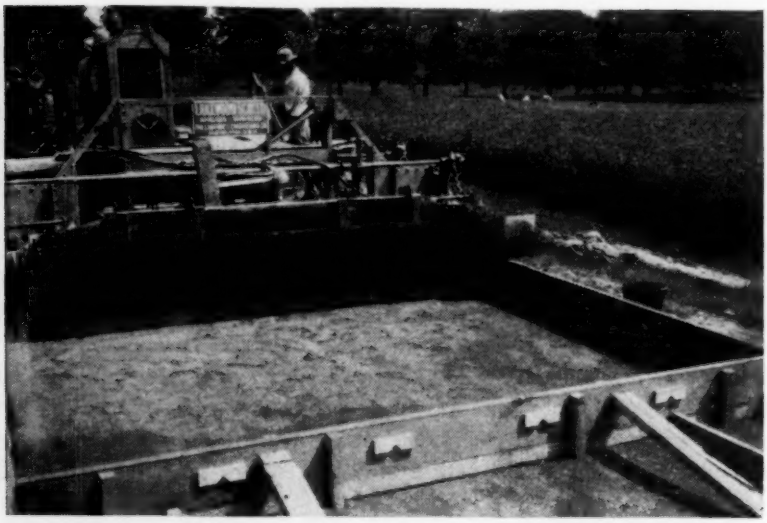


FIG. 5—TEST SECTION JUST BEFORE CONCRETE WAS DEPOSITED, 10-INCH DEPTH

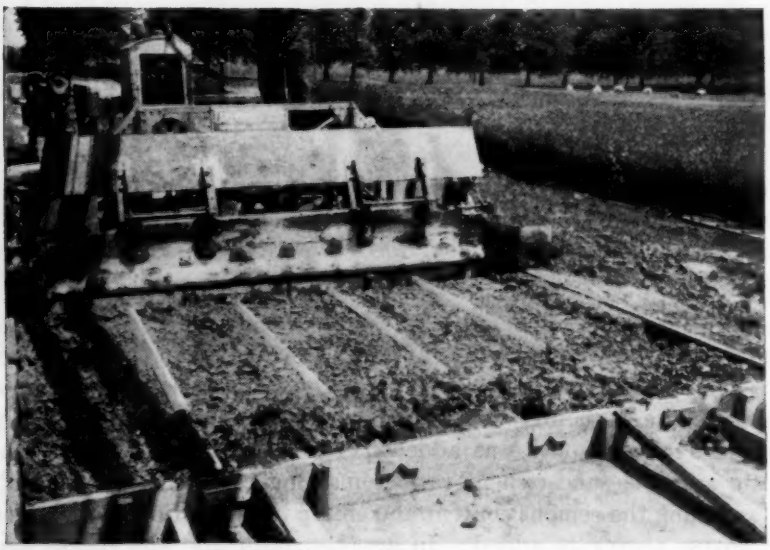


FIG 6—TEST SECTION AFTER FIRST BATCH WAS STRUCK OFF AND WOODEN SEPARATORS INSTALLED, 7-INCH DEPTH

In the third, the cement content was held constant and the water-cement ratio and sand-cement ratio reduced.

PROCEDURE

Either six or seven sections were constructed during each day's run. The first three included (1) the base mix, standard finish, at approximately 2½ in. slump, (2) the base mix, standard finish, reduced to about 1 in. slump by lowering the water-cement ratio, and (3) the same mix and consistency as (2) with the concrete vibrated. The proportions used in the balance of the sections in each day's run were variations of the base mix, all of which were vibrated. It will be observed that the base mix, standard finish, in terms of which all of the other sections were rated, was repeated each day. In this way it was possible to eliminate from the comparisons the effect of differences in quality due to variable weather conditions during the progress of the work. It is believed that this procedure, although it added considerably to the volume of the work, materially increased its value.

METHOD OF DEPOSITING CONCRETE

Departing from the practice formerly employed, the concrete was deposited in two layers in a manner similar to that used when mesh reinforcing is to be installed. The longitudinal wooden separators were placed in position after the first batch was deposited and spread to approximatey one-half the depth of the section. The second batch was then deposited and spread in the usual way. In Series A each bucket was dumped by moving it laterally across the end of the section farthest from the mixer. The concrete was spread with the finishing machine, using a special strike-off for the first layer. This method of dumping the concrete was criticized as not being comparable to the usual field practice. Consequently, in Series B, this part of the procedure was changed by dumping each batch in the center of the section at the end farthest from the mixer and using the bucket to spread the concrete towards the mixer. This spread the concrete to a width of approximately 6 feet and was followed by hand shoveling both ways to the forms with final strike-off by the machine. Fig. 5 is a view of the subgrade just before placing the concrete. Fig. 6 shows the first batch struck off with separators in place. Fig. 7 shows the second batch as deposited by the bucket but before spreading.

TESTING

The 24-in. slabs were tested as simple beams with the load applied at the third points of a 54-in. span. Details regarding the apparatus and methods will be found in earlier reports.[2] The specimens were approximately 9½ months old when tested.

[2] *Public Roads*, Vol. 12, No. 6, Aug., 1931.

FIG. 7—POSITION OF CONCRETE AFTER SECOND BATCH WAS DEPOSITED BUT BEFORE SPREADING—SERIES B

Five cores, 6 in. in diameter, were drilled from each test section, three for density and absorption tests and two for crushing strength tests. Specimens tested for density were drilled from the center and from the two end slabs. In the case of the cores for strength tests, one was drilled from the center and one from an end slab. All cores were drilled from slabs remaining after the flexure tests were made, care being taken to select concrete free from honeycomb for this purpose. Absorption values were computed on the basis of 5 hours' immersion in boiling water. Compression and absorption tests were made at the age of approximately 14 months. All strength specimens were in a saturated condition when tested.

The percentage of honeycomb in the bottom surface and at the cross-section where failure in flexure occurred was determined for each test slab. In general, the same procedure was used for measuring the amount of honeycomb as was employed in the earlier work. For the purpose of determining the rating of each section, an average of the 10 determinations, 5 on the bottom and 5 at the break, was obtained. These are the values which are indicated in the various charts.

Control specimens consisted of 7 x 7 x 30-in. beams and 6 x 12-in. cylinders. None of the control specimens was vibrated. They were all

job cured and were the same age as the pavement slabs and the cores when tested. All specimens were tested wet.

DISCUSSION

The general effect of vibration on the strength and uniformity of the concrete, as revealed by the results secured in Series B with machines A and B and with both crushed stone and gravel, is shown graphically in Fig. 8. The average results of tests on 26 sets of 6 or 7 sections each, involving the standard operation of the vibrators, are given in this figure. Standard operation in the case of machine A is defined as two passes of the machine, with both screeds vibrating. In the case of machine B, standard operation is defined as one pass forward and back with pan vibrating, and a second pass without vibration.

In discussing the figure, the several comparisons which can be made will be considered in the order indicated below.

(*1*) *Effect of Changing Slump to One Inch—No Vibration—Corresponding Reduction in Water-Cement Ratio = 0.09.* These values are shown in the second panel in each of the four blocks. In two groups the average flexural strength was increased approximately 5 per cent by reducing the slump to about 1 in. with a corresponding decrease in water-cement ratio. In the other two groups, one of which represents tests on the 10-in. sections, a slight decrease is noted. A weighted average value for the 22 sections of 7-in. concrete indicates an average increase of about 3 per cent.

The average increase in strength of the corresponding 22 sets of control beams was approximately 7 per cent, a value which may be said to represent the normal increase in strength of the concrete when tested in the usual manner; that is, in the form of 7 x 7-in. beams molded in accordance with standard laboratory procedure.

The smaller increase in strength shown by the slabs as compared to the beams is probably due to the lack of workability of the 1-in. slump concrete, as placed in the pavement. This has been noted in previous reports of similar work.

The lack of workability of the 1-in. slump concrete is also revealed by the high percentage of honeycomb found in the slabs. (See second panels in lower portion of each block.) In every case the tendency to honeycomb was markedly increased by decreasing the slump to 1 in. without vibrating the concrete. This same trend was noted in the earlier work and furnished the justification for the Bureau's requirement that 2-in. slump concrete be used in pavement construction.

Decreasing the slump to 1 in. slightly increases the average variation in slab strength in all three groups of 7-in. slabs and substantially increases the average variation in the case of the 10-in. slabs.

By average variation is meant the average percentage variation in flexural strength of the five slabs composing each test section. It is a measure of the uniformity of strength of the concrete within the section.

For the 22 7-in. sections an increase of about 6 per cent in core strength was obtained as the result of reducing the water-cement ratio. The corresponding average increase in cylinder strength was about 14 per cent which is just about the amount which would theoretically be expected from a change in the water content of this magnitude.

The reason the cores do not show a corresponding increase is not clear. They were all drilled from nonhoneycombed areas and it was expected that they would reflect the effect of changing the water content to about the same extent as the cylinders. However, the cylinders were rodded in accordance with standard laboratory practice whereas the standard finish 1-in. slump slabs may not have been adequately consolidated.

(*2*) *Effect of Vibrating 1-in. Slump Concrete—Same Proportion as Base Mix.* These values are shown in the third panel of each block. Increase in flexural strength as compared to the base mix averaged about 8 per cent for the 22 7-in. sections, or 5 per cent higher than the nonvibrated 1-in. slump concrete. The 10-in. slabs show just about the same average increase. However, in this group, the strength of the 1-in. slump nonvibrated concrete was slightly less than that of the base mix. It is probable that in the case of the 10-in. slabs the lack of workability of the 1-in. slump concrete resulted in less uniform concrete under the standard method of finishing than in the 7-in. slabs. This is indicated by the relatively high variations in slab strengths shown for the 1-in. slump nonvibrated concrete.

The uniformity of the concrete in the three groups of 7-in. slabs as revealed by the percentage of honeycomb was improved somewhat by vibration. In each of these groups, the average amount of honeycomb is slightly less than that shown by the base mix. In the case of the 10-in. slabs it is slightly greater. However, in all cases the amount of honeycomb in the 1-in. slump vibrated concrete is very much less than in the similar mix finished by the usual method. The comparatively large amount of honeycomb found in the 1-in. slump vibrated concrete in the group of 10-in. slabs may indicate that, for

this type of section, a slump somewhat greater than 1 in. would probably prove more satisfactory for vibration. This is indicated by the improvement in uniformity shown in the vibrated sections in which the average slump was 2.2 in. (panel 4). The very substantial reduction in the tendency of the 1-in. slump concrete to honeycomb on the bottom when vibrated seems to indicate definitely that the effect of the surface vibration of the concrete extended entirely through the slabs. This has been a disputed point ever since this method was introduced.

Except for the 10-in. slabs, the average variation in slab strength is about the same as found for the base mix. The 10-in. slabs show considerably less uniform results for the drier mixes.

The crushing strength of the cores from the 22 7-in. sections of 1-in. slump concrete, vibrated, averages about 9 per cent higher than the base mix and about 3 per cent higher than the 1-in. slump nonvibrated concrete. The crushing strength of the cores from the 10-in. sections shows about the same relative improvement. The increase in strength over the 1-in. slump nonvibrated concrete is probably due to vibration. This is indicated by the fact that the control cylinders which, of course, were not vibrated, have almost exactly the same average crushing strength as the control cylinders representing the nonvibrated concrete of the same consistency.

UNIFORMITY AS MEASURED BY DAY-TO-DAY VARIATIONS IN STRENGTH

Further evidence of the undesirability of placing 1-in. slump concrete by the usual methods of finishing is afforded by the following comparison. The average percentage of variation in slab strength shown in Fig. 8 indicates the average uniformity of the concrete within a given test section. Uniformity may also be studied by comparing the average variation from the average strength of the 22 sections of 7-in. concrete containing the base mix with similar variations for the corresponding 22 sections containing the 1-in. slump concrete, both nonvibrated and vibrated. Although these 22 sets of sections were laid on different days, they were laid, as has been noted, in such a way as to eliminate the effect on these comparisons of variations in curing conditions. That is, for each 2½ in. slump standard finish section placed on any one day there was also placed a 1-in. slump standard finish section and a 1-in. slump vibrated section. Average variations in slab strength computed on this basis as well as corresponding variations in the strength of the control beams, the cores and the cylinders are given in Table 2. This table also shows the average strengths of the concrete in these 22 sections.

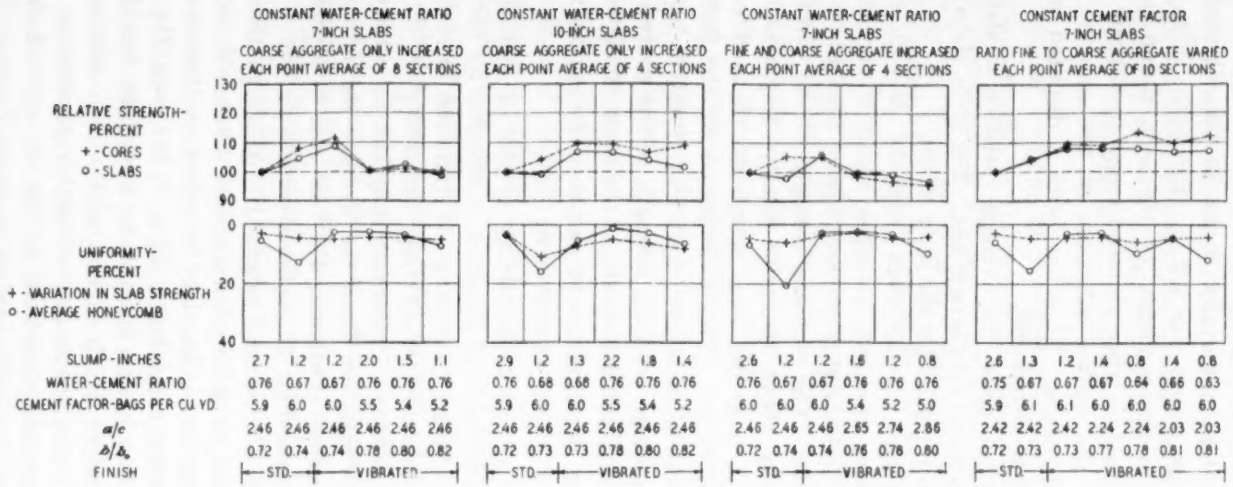

CONSTANT WATER-CEMENT RATIO
7-INCH SLABS
COARSE AGGREGATE ONLY INCREASED
EACH POINT AVERAGE OF 8 SECTIONS
CONSTANT WATER-CEMENT RATIO
10-INCH SLABS
COARSE AGGREGATE ONLY INCREASED
EACH POINT AVERAGE OF 4 SECTIONS
CONSTANT WATER-CEMENT RATIO
7-INCH SLABS
FINE AND COARSE AGGREGATE INCREASED
EACH POINT AVERAGE OF 4 SECTIONS
CONSTANT CEMENT FACTOR
7-INCH SLABS
RATIO FINE TO COARSE AGGREGATE VARIED
EACH POINT AVERAGE OF 10 SECTIONS
RELATIVE STRENGTH-PERCENT
+ - CORES
o - SLABS
130
120
110
100
90
UNIFORMITY-PERCENT
+ - VARIATION IN SLAB STRENGTH
o - AVERAGE HONEYCOMB
0
20
40
SLUMP - INCHES
WATER-CEMENT RATIO
CEMENT FACTOR-BAGS PER CU YD
a/c
b/b0
FINISH
STD.
VIBRATED
2.7 1.2 1.2 2.0 1.5 1.1
0.76 0.67 0.67 0.76 0.76 0.76
5.9 6.0 6.0 5.5 5.4 5.2
2.46 2.46 2.46 2.46 2.46 2.46
0.72 0.74 0.74 0.78 0.80 0.82
2.9 1.2 1.3 2.2 1.8 1.4
0.76 0.68 0.68 0.76 0.76 0.76
5.9 6.0 6.0 5.5 5.4 5.2
2.46 2.46 2.46 2.46 2.46 2.46
0.72 0.73 0.73 0.78 0.80 0.82
2.6 1.2 1.2 1.8 1.2 0.8
0.76 0.67 0.67 0.76 0.76 0.76
6.0 6.0 6.0 5.4 5.2 5.0
2.46 2.46 2.46 2.65 2.74 2.86
0.72 0.74 0.74 0.76 0.78 0.80
2.6 1.3 1.2 1.4 0.8 1.4 0.8
0.75 0.67 0.67 0.67 0.64 0.66 0.63
5.9 6.1 6.1 6.0 6.0 6.0 6.0
2.42 2.42 2.42 2.24 2.24 2.03 2.03
0.72 0.73 0.73 0.77 0.78 0.81 0.81

FIG. 8—GENERAL AVERAGE VALUES—STANDARD OPERATION OF VIBRATORS—SERIES B—EACH POINT IS THE AVERAGE OF SECTIONS CONTAINING CRUSHED STONE AND GRAVEL FINISHED WITH MACHINES A AND B

TABLE 2—AVERAGE STRENGTH AND AVERAGE VARIATION IN STRENGTH
SERIES B—STANDARD OPERATION OF VIBRATORS

(a) Modulus of Rupture

Finish	Consistency (Slump)	Pavement Slabs			Control Beams[1]		
		Average Strength	Average Variation in Strength	Increase over Base Mix	Average Strength	Average Variation in Strength	Increase over Base Mix
	Inches	p.s.i.	Per Cent	Per Cent	p.s.i.	Per Cent	Per Cent
Standard	2½	749	2.9	–	635	4.4	–
Standard	1	772	4.4	3	682	4.4	7
Vibrated	1	807	2.6	8	—	—	–

(b) Crushing Strength

Finish	Consistency (Slump)	Pavement Cores			Control Cylinders[2]		
		Average Strength	Average Variation in Strength	Increase over Base Mix	Average Strength	Average Variation in Strength	Increase over Base Mix
	Inches	p.s.i.	Per Cent	Per Cent	p.s.i.	Per Cent	Per Cent
Standard	2½	6,860	4.6	–	5,540	5.2	—
Standard	1	7,250	4.2	6	6,310	4.7	14
Vibrated	1	7,490	4.2	9	6,340	4.3	14

[1]Each strength value is the average of tests made in connection with the construction of the 22 sections of 7-in. concrete in Series B which involved the standard operation of vibrating equipment and is the weighted average of values obtained with machines A and B and with crushed stone and gravel.
[2]Control specimens molded in accordance with standard laboratory methods.

It will be observed that the average percentage variation in slab strength of the 2½-in. slump standard finish concrete (2.9) is considerably less than the corresponding value for the 1-in. slump standard finish (4.4) and slightly greater than the 1-in. slump vibrated concrete (2.6). Th relatively low variations for the control specimens are also of interest in view of the comparatively wide range in temperature and humidity which occurred during the construction period. The 22 sets of sections shown in the table were constructed between July 19 and Sept. 6, during which time the temperature for the actual construction period ranged from a maximum of 98°F. on Aug. 7 to a minimum of 71° F. on Aug. 13 and 21, and the relative humidity ranged from 75 per cent on Aug. 16 to as low as 28 per cent on Aug. 27 and 31.

It may also be noted that the average day-to-day variation in flexural strength of the 2½-in. slump standard finish slabs was just about the same as the average variation in the strength of the five slabs composing a given section (Fig. 8). This would indicate that variations in curing and other conditions incidental to the average job do not cause variations in the strength of sections laid on different days to any greater degree than those factors such as depositing and spreading which affect the uniformity of the concrete within a given section.

(3) *Effect of Vibrating Mixes Harsher Than Base Mix.* (a) *Constant W/C—Coarse Aggregate Only Increased.* These values for the 7-in. sections are shown in the fourth, fifth and sixth panels of the first block of Fig. 8. It will be observed that the average flexural and compressive strengths are both almost exactly the same as the base mix, nonvibrated. The uniformity of the concrete, as measured by the percentage of honeycomb is also just about the same, although there is a tendency for the amount of honeycomb to increase as the mix becomes harsher. Uniformity as measured by variation in slab strength is about the same for all conditions in this group.

Corresponding values for the 10-in. slabs are shown in the second block of Fig. 8. In this case all of the harsher mixes when vibrated show higher strengths both in flexure and compression than the base mix, although the flexural strength decreases as the percentage of coarse aggregate is increased. The uniformity of the harsher mixes as revealed by honeycomb also compares favorably with the standard although we find a decrease in uniformity as the amount of coarse aggregate is increased beyond a certain point. Uniformity as indicated by variations in slab strength is somewhat less than for the corresponding 7-in. sections.

In the case of both the 7-in. and the 10-in. slabs the leanest and harshest mixes gave as high strength as the base mix even though the cement factor averaged 3/4 sack per cu. yd. less. The slump averaged a little more than 1 in. and the water-cement ratio and sand-cement ratio were the same as for the base mix, that is, 0.76 and 2.46, respectively.

(b) *Constant W/C—Fine and Coarse Aggregate Increased.* These results are shown for the 7-in. slabs in the third block in Fig. 8. The results do not differ materially from those shown in the first block, although there is a tendency for a slightly greater reduction in strength as the quantities of the aggregates are increased. The percentage of honeycomb was substantially increased for the sections in the group showing less than 1-in. slump, indicating that this value is about the lowest that should be used with the type of equipment employed in these tests. This holds true even where the cement content is maintained constant. (See fifth and seventh panels in the fourth block.) It will be noted that the harshest mix in group 3 contained somewhat less cement and had a lower slump than the corresponding sections in groups 1 and 2. This was due to the increased sand content. The results indicate that for vibration of this type the slump should not be less than 1 in. and that the cement factor should not be reduced more than ¾ sack per cu. yd. They also show that

nothing is gained either in strength or uniformity by increasing the sand content.

(c) *Constant Cement Factor—Ratio of Fine to Coarse Aggregate Varied.*—In this case it will be observed that the strength of all the vibrated sections is greater than the base mix. This applies to both flexure and compression. The flexural strength remains virtually the same but the crushing strength is increased in the two cases where the slump was reduced to 0.8 by lowering the water-cement ratios to 0.64 and 0.63, the lowest average values used in the test. It would appear from these data that the flexural strength of the concrete is not affected appreciably by varying the ratio of fine to coarse aggregate within the limits used in these tests. However, the tendency to honeycomb is increased by using less than a 1-in. slump. The comparatively high crushing strengths of the sections containing the 0.8-in. slump concrete is explained by the fact that the cores were drilled from nonhoneycombed areas. Under these conditions the strengths paralleled the reduction in water-cement ratio. These data illustrate the danger of drawing conclusions regarding the quality of paving concrete solely from the results of core tests.

Effect of Type of Coarse Aggregate and Type of Finishing Machine. The data shown in Fig. 8 include results obtained with crushed stone and gravel concrete finished with machines A and B. A study of the detail data from which this figure was prepared indicates that the two types of aggregate gave about the same results insofar as improvement in strength and uniformity due to vibration are concerned. From the standpoint of effect of type of finishing machine, the data are not consistent. Certain data indicate one type to be somewhat more effective than the other. However, other tests indicate the reverse to be true. In general, the two types of machine appear to be about equally effective.

Specific Gravity and Absorption. The average values for bulk specific gravity and absorption of 6-in. concrete cores drilled from sections in Series B are shown in Fig. 9. The same grouping is used as in Fig. 8. These values reflect in a general way variations in density of the concrete, as revealed by tests on portions of the slabs free from honeycomb. They bear no relation necessarily to the average density of the slabs as a whole, nor to the density of any portion in which honeycomb areas may be included. The differences shown are not of great magnitude in any case, although there seems to be a slight tendency for increased specific gravity and lower absorption in the case of the sections which were vibrated. It will also be observed

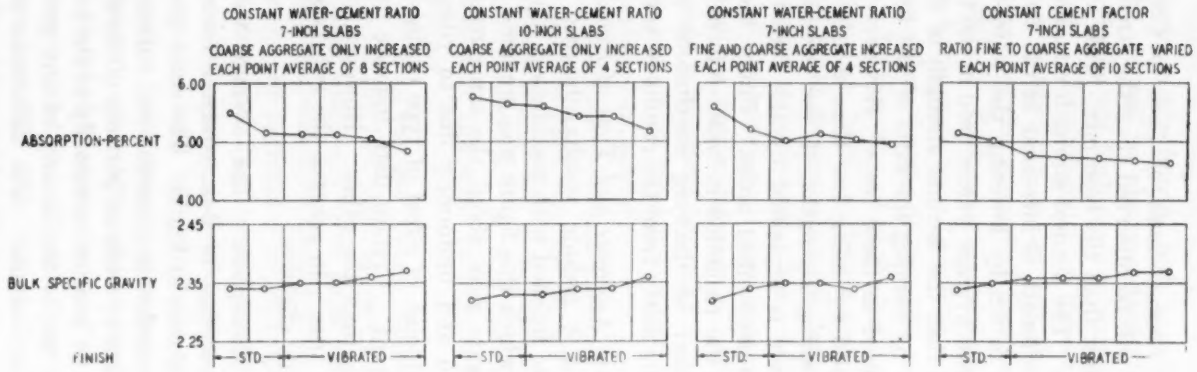


FIG. 9—GENERAL AVERAGE VALUES FOR ABSORPTION AND SPECIFIC GRAVITY—STANDARD OPERATION OF VIBRATORS—SERIES B—EACH POINT IS THE AVERAGE OF SECTIONS CONTAINING CRUSHED STONE AND GRAVEL FINISHED WITH MACHINES A AND B. SECTIONS READING FROM LEFT TO RIGHT SAME AS IN FIGURE 8

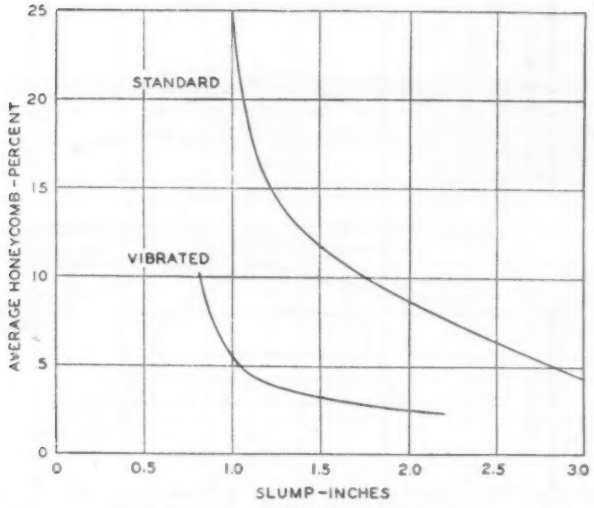


FIG. 10—RELATION BETWEEN HONEYCOMB AND SLUMP—SERIES B—7-INCH SLABS—AVERAGE OF STONE AND GRAVEL, MACHINES A AND B—STANDARD OPERATION OF VIBRATORS

that in general density increases as the relative proportion of coarse aggregate in the concrete increased.

As was indicated in the previous report, it is probable that the harsher concretes, when freshly mixed, contained more air voids than the base mix. The fact that the final densities of these harsh mixtures, as revealed by bulk specific gravity tests on the cores, tend to arrange themselves in about the same order as the theoretical densities (ratio of the sum of the absolute volume of solids to volume of concrete) would indicate that the increased densities corresponding to the changes in composition were actually being obtained as the result of the method of compaction used.

GENERAL DISCUSSION OF SERIES B

It will be evident from the preceding discussion that one of the benefits derived from the use of vibration in these tests was the improvement in the uniformity of the 1-in. slump concrete obtained by reducing the amount of honeycomb in the slabs. The general relation between honeycomb and slump is shown in Fig. 10. It will be observed that the tendency of the standard finish concrete to develop honeycomb increased rapidly in the region of 1-in. slump and that a slump of about 3 in. was required to reduce the honeycomb to 5 per cent. On the other hand, the same degree of uniformity was obtained with the vibrated concrete at about 1-in. slump.

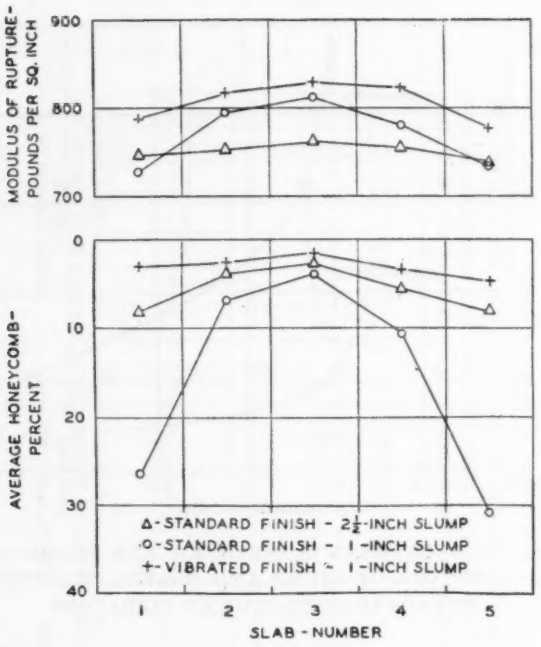


FIG. 11—RELATION BETWEEN SLAB STRENGTH AND HONEYCOMB—STANDARD OPERATION OF VIBRATORS—7-INCH SLABS—SERIES B—AVERAGE OF MACHINES A AND B, STONE AND GRAVEL

It will be recalled that in this series the concrete in the two outside slabs was distributed by shovels from the mass as deposited by the mixer bucket. This practice of distributing the concrete along the forms and in the corners formed by joints, etc., is practiced to a certain extent, although efforts are made on well-conducted jobs to eliminate as much of this hand work as possible.

The effect of the location of the test slab in the section on the strength and amount of honeycomb in the concrete is shown in Fig. 11. The upper portion of the chart gives for each of the five slabs the average flexural strength of the three groups of 22 7-in. sections shown in Fig. 8, in which the base proportions were used. The lower portion of the chart shows the corresponding percentages of honeycomb in the slabs. Each point is the average of 22 individual values. This figure is of interest particularly in showing how the nonuniform distribution in slabs 1 and 5 in the 1-in. slump standard finish concrete affected the strength. In both cases the average strength in the two outer slabs was less than the strength of the 2½-in. slump concrete. The effect of vibration on honeycomb is also brought out clearly. Note

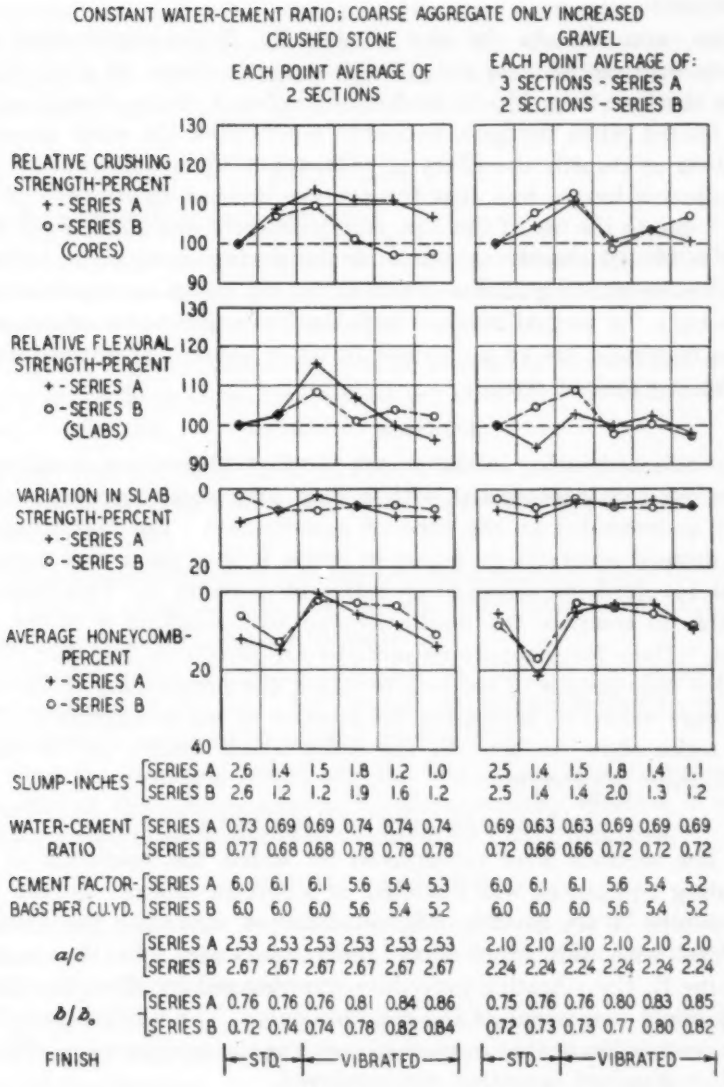

CONSTANT WATER-CEMENT RATIO: COARSE AGGREGATE ONLY INCREASED
CRUSHED STONE
EACH POINT AVERAGE OF 2 SECTIONS
GRAVEL
EACH POINT AVERAGE OF:
3 SECTIONS - SERIES A
2 SECTIONS - SERIES B
RELATIVE CRUSHING STRENGTH-PERCENT
+ - SERIES A
o - SERIES B
(CORES)
130
120
110
100
90
RELATIVE FLEXURAL STRENGTH-PERCENT
+ - SERIES A
o - SERIES B
(SLABS)
130
120
110
100
90
VARIATION IN SLAB STRENGTH-PERCENT
+ - SERIES A
o - SERIES B
0
20
AVERAGE HONEYCOMB-PERCENT
+ - SERIES A
o - SERIES B
0
20
40
SLUMP-INCHES
SERIES A 2.6 1.4 1.5 1.8 1.2 1.0 2.5 1.4 1.5 1.8 1.4 1.1
SERIES B 2.6 1.2 1.2 1.9 1.6 1.2 2.5 1.4 1.4 2.0 1.3 1.2
WATER-CEMENT RATIO
SERIES A 0.73 0.69 0.69 0.74 0.74 0.74 0.69 0.63 0.63 0.69 0.69 0.69
SERIES B 0.77 0.68 0.68 0.78 0.78 0.78 0.72 0.66 0.66 0.72 0.72 0.72
CEMENT FACTOR-BAGS PER CU.YD.
SERIES A 6.0 6.1 6.1 5.6 5.4 5.3 6.0 6.1 6.1 5.6 5.4 5.2
SERIES B 6.0 6.0 6.0 5.6 5.4 5.2 6.0 6.0 6.0 5.6 5.4 5.2
a/c
SERIES A 2.53 2.53 2.53 2.53 2.53 2.53 2.10 2.10 2.10 2.10 2.10 2.10
SERIES B 2.67 2.67 2.67 2.67 2.67 2.67 2.24 2.24 2.24 2.24 2.24 2.24
b/b₀
SERIES A 0.76 0.76 0.76 0.81 0.84 0.86 0.75 0.76 0.76 0.80 0.83 0.85
SERIES B 0.72 0.74 0.74 0.78 0.82 0.84 0.72 0.73 0.73 0.77 0.80 0.82
FINISH
STD.
VIBRATED
STD.
VIBRATED

FIG. 12—COMPARATIVE SLAB STRENGTH AND UNIFORMITY FOR SERIES A AND B. STANDARD OPERATION OF MACHINE A—7-INCH SLABS

that the average percentage of honeycomb in the 22 No. 1 slabs was reduced from 26 to 3 per cent, and in the 22 No. 5 slabs from 31 to 4 per cent.

Also, note that in the case of slab No. 3 the nonvibrated 1-in. slump concrete showed an average strength about 50 p.s.i. higher than the base mix, 2½-in. slump. Slab No. 3, being directly under the bucket when dumped, probably represented the most favorable location as regards the effect of consistency upon uniformity. This is indicated by the fact that the average increase in strength of slab No. 3 due to the use of the 1-in. slump concrete was about 7 per cent. This is almost exactly the same as the average increase in strength of the corresponding 22 sets of control beams, which as noted probably represents the normal increase in flexural strength to be expected for these materials due to a decrease in water content of approximately ¾ gal. per sack of cement.

SERIES A—MACHINE

As indicated earlier in the report, the first 42 sections, constituting series A, were constructed with a base mix which appeared somewhat undersanded at the time of construction. For this reason it was deemed advisable to repeat in Series B that portion of Series A which involved the standard operation of machine A. This makes it possible to compare the results secured with machine A in the two series. Data for comparable sections are shown in Fig. 12. They involve only groups of sections in which the proportions of the base mix were varied by increasing the amount of coarse aggregate. The two series show, in general, the same trends insofar as the effect of vibration is concerned.

SPECIAL OPERATION OF MACHINES A AND B

A few sections were constructed on which the operation of the vibrating equipment was modified to a certain extent. In the case of machine A the modification consisted of operating the finishing machine with the front screed only vibrating. In the case of machine B, the vibrating procedure was changed by vibrating during the forward movement of the machine only. The limited data from these tests indicate that in neither case was the equipment as efficient as when standard operation was employed.

In addition to the data obtained with machines A and B, two other pieces of vibratory equipment were investigated to a very limited extent. One was a comparatively light hand screed designed for locations where a standard finishing machine cannot be used. The other was a pan vibrator resting directly on the concrete and

operating in a manner similar to machine B. This machine was designed for use with a finishing machine not equipped with vibrators. The results obtained with the pan vibrator compared favorably with those obtained with machines A and B. On the other hand the limited data obtained with the hand screed indicated that it was not quite so effective as either machines A or B, or the pan vibrator.

CONCLUSIONS

The data from these tests indicate:

1. That for a given water-cement ratio a saving of approximately 10 per cent in the amount of cement may be effected by the use of vibration without sacrificing strength and uniformity.

2. That for a given cement content an increase in flexural and compressive strength of approximately 10 per cent may be obtained by the use of vibration.

3. That with the methods of surface vibration investigated the best results will be secured with a consistency corresponding to a slump of approximately 1 in.

4. That the uniformity of concrete having a slump of approximately 1 in. (that is, freedom from segregation, honeycomb areas, etc.) is markedly improved by surface vibration.

5. That the effect of surface vibration on strength and uniformity seems to be about the same for concrete containing gravel as coarse aggregate as for concrete containing crushed stone.

6. That the vibrating screed and the vibrating pan mounted independently of the screeds seem to be about equally effective in producing the results enumerated in Conclusions 1 to 5, inclusive.

PRACTICAL APPLICATION

Depending upon the objective sought, existing specifications for pavement concrete may be modified to utilize vibration to advantage in either of the following ways:

1. By providing for adjustment of proportions to give a slump of approximately 1 in. with the same net water-cement ratio as used in standard construction.

2. By providing for adjustment of proportions to give the same cement content as used in standard construction but with the slump specified at 1 in. instead of 2½ in.

In either case the specification should be worded so as to permit the engineer to vary the relative proportions of fine and coarse aggre-

gate to produce the best results, depending upon the type and grading of the aggregates used and the type of finishing equipment employed.

The Bureau desires to express its appreciation of the courtesy extended by the following companies in loaning the vibratory equipment used in these tests: Jaeger Machine Co., Columbus, Ohio; Blaw-Knox Co., Pittsburgh, Pa.; Baily Vibratory Co., Philadelphia, Pa.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by June 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MARCH-APRIL 1937

The New Federal Specifications for Portland Cements*

By Ben Moreell
MEMBER AMERICAN CONCRETE INSTITUTE

Until recently, we have had but one generally accepted standard specification for portland cement. While the cements of some manufacturers were recognized as superior in certain respects to those of others, engineers agreed, in general, that if the cement met the requirements of this specification it was "pretty good stuff" and could be used safely for practically any purpose.

As far as I know, the earliest American specification for portland cement, which might be referred to as a "standard" by reason of the extent to which it was used, was promulgated in 1903 by the American Railway Engineering Association. It defined portland cement as "a product of the mixture of clay and lime carbonate in definite proportions, calcinated at a high temperature and reduced to a fine powder." This specification was prepared primarily for use by the railroads and was, I believe, not intended as an American standard.

In 1904, Committee C-1 of the American Society for Testing Materials, under the Chairmanship of the late Professor Swain, recommended a specification for "Natural and Portland cements." In discussing this specification, the *Engineering News* of June 30, 1904, stated "Its importance arises mainly from the fact that this society aims to produce specifications which shall be, as nearly as possible, standards for this country."

This specification defined portland cement as "the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcarious materials, and to which no addition greater than 3 per cent has been

*Presented 33rd Annual Convention, American Concrete Institute, New York, Feb. 23-26, 1937.
†Commander, C. E. C., U. S. Navy.

made subsequent to calcination." The specification was adopted by the society in 1904. It was revised in 1908, 1909, 1916, 1920, 1926 and 1930, but when consideration is given to the span of years and the revolutionary changes in other fields of technical and industrial activity, it is truly remarkable that the current A. S. T. M. specification for portland cement bears such a close resemblance, both in definition and in detail, to its ancestor.

The first U. S. Government specification for portland cement was prepared by a committee of Government engineers, organized for this purpose in 1911, and consisting of representatives of all cement-using departments of the Government. The committee recommended a specification which was adopted by a Departmental Conference in 1912, and was promulgated by Executive Order.

It is interesting to note that 25 years ago a new cement specification for the Federal Government was an event of such transcending importance that only an order signed by the President of the United States was considered a vehicle fitting for its promulgation.

This first Government specification did not differ materially in its definition or in its requirements from the A. S. T. M. specification of 1904, although the Government specification was somewhat more complete. Since the adoption of this specification there have been numerous revisions, but the Government and the A. S. T. M. specifications have, in general, kept in step and until recently there have been no pronounced differences.

Until 1930, the A. S. T. M. and the Federal Government adhered to the so-called "single-standard," that is, there was only one "standard" portland cement. Even prior to 1930, however, there were rumblings of dissatisfaction with this situation. The discontent resulted, apparently, from the slowly awakening realization among engineers that although there was but one standard specification for portland cement, this by no means indicated that all portland cements were equal. Studies of the comparative values of cements showed that even though all could pass the standard specification, there were variations of considerable magnitude in strength, durability, workability, heat generation, and other properties. The result was a movement to specify, in more detail than the standard specification, the special property required for the project in hand.

At first, the most insistent demands came from those who needed a cement with high strength at early ages. The introduction of "high alumina" cement on a commercial scale encouraged new uses for cement with high early strength and stimulated studies of the economy of such cement. The demands of potential users and the

prospective competition from makers of high alumina cement influenced the portland cement manufacturers to develop a high early-strength portland cement which could be sold at a small premium over the standard cement.

In 1930, the A. S. T. M. adopted a tentative specification for high early-strength portland cement. In 1936, this specification was adopted in amended form as a "standard." It is interesting to note that in this specification the formal heading entitled "definition" is omitted for the first time since the original specification of 1904. Although the specification does contain a definition under the heading of "chemical limits," it differs materially from the definition given in the current A. S. T. M. specification for normal portland cement.

The adoption of an additional "standard specification" for portland cement was not accomplished without controversy and some opposition on the part of the industry. It should be noted here that this opposition was, in a measure, justified. It is unquestionably true that the wholesome and rapid development of cement and concrete in this country can be attributed in large part to the existence of the "single standard", which has enabled the manufacturer to devote his energies to the development of manufacturing and business procedures. This, in turn, has resulted in low prices and consequent tremendous expansion in the use of cement.

While the demand for high early-strength cement resulted from the desire for high-speed production, there began to appear other special cements advertised to have special properties, resulting either from the character of the raw materials, from the peculiarities of the manufacturing process, or from additions during or after grinding. Engineers demanded low-heat cements for mass construction and sulphate-resisting cements for severe alkali exposures. Highly plastic cements, waterproofed cements, high-yield cements, and others with liberally-advertised special properties came on the market in increasing numbers.

In a paper on "Recent Trends in European Concrete Practice" delivered before the Portland Cement Association two years ago, I offered some comment on this situation, which may be pertinent to this discussion. In speaking of French practice, I stated:

> The user of hydraulic cements has the choice of a wide variety of materials and grades.
>
> Table 1 shows the classification of hydraulic cements proposed in 1918 by the French Standardization Commission and adopted in 1927 by the Ministry of Public Works. All of the hydraulic cements listed are included under the same standard specification. You will note that there are 13 different grades, not counting the

TABLE 1—CLASSIFICATION OF HYDRAULIC CEMENTS
By French Ministry of Public Works—December 24, 1927

Nature of Cement	Grade	Tensile Strength, ½ Briquette—p.s.i.	
		7 Days	28 Days
A—Artificial (Portland)	A—Standard	142	213
	B—For reinforced work	213	284
	C—For special purposes	284	355
B—Natural	A—Standard	43	71
	B—Special	85	142
C—Grappiers	A—Standard	85	142
	B—Special	142	213
D—Mixed*	——	——	——
E—Puzzolan* slag cement	A—Standard	85	142
	B—Superior	142	213
	C—Special	213	284
F—Hydraulic lime	A—Standard	14	43
	B—Strongly hydraulic	43	71
	C—Specially hydraulic	85	142

*Mixed cements (Grappiers ground with natural or Portland cements), natural cements, and puzzolan cements are made under such varying conditions and with consequent varying qualities that the specification for the work should specify the strength limit which in no case should fall below 85 and 142 p.s.i. at 7 and 28 days, respectively.
Slag cement is a particular variety of puzzolan cement.

unlisted varieties of mixed cements. The principal classification, by "Nature of cement," lists six varieties. The secondary classification, designated as the "grade," classifies the cements by its qualities, i.e., time of set and strength.

It is interesting to speculate on the reasons for this wide variety of recognized hydraulic cements, all included under one specification. In my opinion, there are three principal reasons:

First, and most important, is an economic reason. Material costs are a much larger percentage of the total project cost than in the United States. Therefore, any saving in cement cost will be of greater relative importance. The French engineer is not only concerned that the cement shall be good enough for the work in hand but he is equally concerned that it shall not be better than is absolutely required. Price is an important consideration.

The second reason is that the opportunity to exercise a choice from a wide variety of cements is peculiarly in keeping with French engineering psychology, which encourages originality and initiative, the use of new materials and methods, and new applications of old materials and methods.

The third reason, also economic, is that the Government wishes to encourage the widest possible use of all of the natural resources of the country, as a measure of national economy.

In August 1933, the French Standardization Society presented a new specification for hydraulic cements. This specification includes nine groups, comprising in all 36 different grades. It is evident from the data presented in Table 2 that several of the grades are over-lapping and that the unskilled purchaser is likely to buy

strictly on a *price* basis, rather than on the basis of comparative qualities.

It is interesting to note that in the United States the current demand for cements with special qualities arises, not from a desire to obtain a lower-priced cement, but

TABLE 2—CLASSIFICATION OF HYDRAULIC CEMENTS
By French Standardization Society—August, 1933

Nature of Cement	Class	Grade	Compressive Strength, ⅓ Cube—p.s.i.		
			48 Hours	7 Days	28 Days
I—Limes, Hydraulic	A—Hydraulic limes	1—Hydraulic	——	71	178
		2—Strongly hydraulic	——	178	355
		3—Strongly hydraulic for special purposes	——	450	900
	B—Hydraulic lime—slag blend	1—Standard	——	178	355
		2—Special	——	450	900
II—Natural cements	A—Rapid setting	1—Ordinary	178	285	450
		2—Standard	285	450	710
		3—Special	450	710	1135
	B—Slow setting	1—Ordinary	——	178	355
		2—Standard	——	355	710
		3—Special	——	710	1420
III—Grappier	A—Grappier	1—Ordinary	——	178	355
		2—Standard	——	355	710
		3—Special	——	710	1420
IV—Blended cements	A—Mixed.. / B—Diluted.	1—Ordinary	——	178	355
		2—Standard	——	355	710
		3—Special	——	710	1420
	C—Pussolan	1—Standard	——	1135	2270
V—Portland cements	A—Portland	1—Standard	——	1420	2270
		2—Special	——	2270	3550
	B—High early strength	1—Standard	1420	3550	4500
		2—Special	2270	4500	5680
VI—Slag cements	A—Lime base	1—Standard	——	710	1420
		2—Special	——	1420	2270
	B—Portland cement base	1—Standard	——	710	1420
		2—Special	——	1420	2270
VII—Metallurgical cements	A—High Portland—low slag	1—Standard	——	1420	2270
		2—Special	——	2270	3550
	B—Low Portland—high slag	1—Standard	——	1420	2270
		2—Special	——	2270	3550
VIII—High sulphate slag cements	——	1—Standard	——	1420	2270
	——	2—High early strength	1420	3550	4500
IX—High alumina cement	——	——	4500	5050	5680

because the engineer feels that cement which meets the requirements of the standard specification will not necessarily meet the special requirements of his particular project. He is willing to pay as much or more for the special cement if it will do the work for him. It is questionable whether such a variety of hydraulic cements as is indicated by this chart could be justified under American economic conditions. But there is something to be said in favor of a limited number of cements with special qualities which will meet special needs provided, however, that the need is real and the production of a special cement is economically justified. That such conditions do actually exist is attested by the current production of special cements for specific projects of large magnitude.

I have quoted the foregoing to illustrate what I believe will be the inevitable result if the restraining influence of a reasonable measure of "birth control" is not applied to the propagation of cement specifications.

Since, during the past few years, by far the greater part of heavy construction has been performed by the Federal Government, the demand for special cements has been most evident in the Government departments. The Federal Specification Executive Committee will not sanction the preparation of a Government specification until the demand by the Government Departments becomes insistent, and not even then unless it can be shown conclusively that the use to be made of the specification is sufficient to justify the work involved in its preparation. Because of the large quantities of cements already being purchased by the Government departments under non-standard specifications, and to comply with the many requests for specifications for special cements, the Technical Committee on Cement, Lime and Plaster of the Federal Specification Executive Committee, under the able leadership of P. H. Bates, of the National Bureau of Standards, undertook in the fall of 1935 the preparation of such specifications, as well as the revision of the then current specification for normal portland cement.

It was, at first, proposed to prepare one master specification to cover all varieties of hydraulic cements, including the portland-pozzolanic type. However, on further consideration, the Committee concluded that the experimental data and test procedures necessary for the determination of comparative pozzolanic values of the many so-called pozzolanic materials on the market, were as yet not sufficiently established to warrant inclusion in such a master specification. It was decided that, for the time being, the specification would cover portland cements only. It was decided, also, that the inclusion of all types of portland cements under one specification might confuse the user and that a separate specification for each type would encourage frequent revisions as more experience and research data on the special types became available.

The Committee decided, therefore, on the preparation of five separate specifications:

1. Normal portland.
2. High early strength.
3. Moderate heat of hardening.
4. Low heat.
5. Sulphate resisting.

Later, it was found that the differences between the low-heat and moderate-heat cements were not sufficient to justify another type intermediate between the moderate-heat and the sulphate-resisting types. Experience in the use of low-heat cements at Boulder Dam confirmed this view and the low-heat cement was omitted. This left four types, three of which constituted new standards and the fourth a revision of the then existing standard for normal portland cement. In addition, it was decided that the specifications for "methods of sampling, inspection and testing hydraulic cements," which are applicable to all of the cement specifications, would be issued as a separate document.

After consultation with the producers and with the interested Government departments, the Technical Committee recommended to the Federal Specification Executive Committee the adoption of the 4 new cement specifications and the new specification for sampling, inspection and testing, and on Sept. 30, 1936, the specifications were formally adopted. They are now being printed and will be ready for distribution to the public about April 15, 1937.

Table 3 indicates the principal features of these new specifications and, for comparison, the corresponding features of other standards. Before discussing some of the details of the specifications, I wish to state that I am not a cement chemist. My opinions are those of a practicing engineer who has a special interest in cement and concrete. I do not propose to defend the technical committee's decisions. While I am thoroughly in accord with those decisions, I feel that there are others more able than I to defend them; nor do I have authority to speak for the technical committee. Therefore, any explanation of the committee's actions which I may offer should be taken as my own interpretation and not as the committee's views.

The first notable change in the new specifications is the omission of the definition for portland cement. The former Federal Specification contained the same definition as the current A. S. T. M. specification. The new specifications have no definitions, as such, but in lieu thereof, there is the following statement:

> The hydraulic cement purchased under this specification may be made from any selection of raw materials and by any process that will yield a product complying with the detailed requirements of the specification.

TABLE 3—COMPARISON OF CEMENT SPECIFICATIONS

Name of Cement	Chemical Requirements									Specific Gravity (Min.)	Fineness (Min.)	Time of Set		Tensile Strength							Compressive Strength				Heat of Hydration	
														Neat Cement			3-1 Mortar				1-2.75 Mortar					
	Loss on Ignition (Max.)	Insoluble Residue (Max.)	SO_3 (Max.)	MgO (Max.)	Al_2O_3 (Max.)	Fe_2O_3 (Max.)	C3A (Max.)	SiO_2 (Min.)	Ratio Al_2O_3 + Fe_2O_3			Initial (Minutes)	Final (Hours)	1 Day	7 Days	28 Days	1 Day	3 Days	7 Days	28 Days	1 Day	3 Days	7 Days	28 Days	7 Days	28 Days
1904 A.S.T.M. Portland	—	—	1.75	4.00	—	—	—	—	—	3.10	92% Passing 100 Sieve 75% Passing 200 Sieve	≧ 30 (Vicat)	≦ 1 ≧ 10	150–200	450–550	550–650	—	—	150–200	200–300	—	—	—	—	—	—
1912 U.S.Gov't Portland	4.00	1.00	1.75	4.00	—	—	—	—	—	3.10	92% Passing 100 Sieve 75% Passing 200 Sieve	≧ 45 (Gillmore)	≦ 10	—	500	600	—	—	200	275	—	—	—	—	—	—
1930 Federal and A.S.T.M. Portland	4.00	0.85	2.00	5.00	—	—	—	—	—	—	78% Passing 200 Sieve	≧ 45 (Vicat) ≧ 60 (Gillmore)	≦ 10	—	—	—	—	—	275	350	—	—	—	—	—	—
1936 A.S.T.M. High Early	4.00	0.85	2.50	5.00	—	—	—	—	—	—	Specific Surface ≧ 1900 sq. cm. per gm.	≧ 45 (Vicat) ≧ 60 (Gillmore)	≦ 10	—	—	—	275	375	—	Greater than at 3 days	1300	3000	—	Greater than at 3 days	—	—
* Portland	3.00	0.75	2.00	5.00	7.50	6.00	15.00	—	—	—	≧ 1500 sq. cm. per gm.	≧ 60 (Gillmore)	≦ 10	—	—	—	—	175	275	350	—	900	1800	3000	—	—
* High Early	3.00	0.75	2.50	5.00	7.50	6.00	15.00	—	—	—	≧ 1900 sq. cm. per gm.	do	≦ 10	—	—	—	275	375	425	—	1250	2500	3500	—	—	—
* Moderate Heat	3.00	0.75	2.00	5.00	6.00	6.00	8.00	21.00	R ≦ 2.0 R ≧ 0.7	—	≧ 1800 sq. cm. per gm.	do	≦ 10	—	—	—	—	125	250	325	—	750	1500	2500	70	80
* Sulphate Resisting	3.00	0.75	1.75	4.00	4.00	4.00	5.00	24.00	R ≦ 2.0 R ≧ 0.7	—	≧ 1800 sq. cm. per gm.	do	≦ 10	—	—	—	—	—	175	300	—	—	1000	2200	—	—

*1936 Federal Specifications

I will comment briefly on each of the four new specifications, begining with the normal portland cement.

NORMAL PORTLAND CEMENT

Under "Chemical requirements," the "loss on ignition" has been reduced from 4.00 to 3.00 per cent, and the insoluble residue from 0.85 to 0.75 per cent. The sulphuric anhydride and the magnesia remain the same. However, new requirements limit the alumina to 7.50 per cent, the iron oxide to 6.00 per cent and the tri-calcium aluminate to 15.00 per cent.

The following significant note appears under the table of chemical limits:

The expressing of chemical limitations by means of calculated assumed compounds does not necessarily mean that the oxides are actually or entirely present as such compounds.

Under "fineness" an important change has been made. The committee has dropped the outmoded sieve analysis method and has specified that the specific surface as determined by the Wagner turbidimeter shall be not less than 1500 sq. cm. per gram.

The "soundness" requirement remains the same.

The "time of setting" remains the same except that the use of the Gillmore needle is made mandatory.

Under "tensile strength" the requirements remain the same except that a 3-day strength requirement of 175 p.s.i. has been added, as well as a requirement that the strength at 7 and 28 days shall be greater than that at 3 and 7 days, respectively.

The Committee has added a new paragraph on "compressive strengths" which calls for tests on mortar cubes of 1 part cement to 2.75 parts, by weights, of fine testing sand, and requires average strengths of 900, 1800 and 3000 p.s.i. at 3, 7 and 28 days, respectively. The strengths at 7 and 28 days must be higher than at 3 and 7 days, respectively.

The purchaser is given the option of using either tensile or compressive tests or both, but if the compressive tests are not required specifically, the tensile tests only are mandatory.

With respect to age at testing, the specification requires that, unless specified otherwise, the tensile and compressive strength tests will be made at 3 and 7 days only.

The following paragraph, headed "Additions during grinding" is new:

Water and/or any variety of calcium sulphate to which no additions have been made can be used in amounts such that the limits shown in paragraph E-1 for the ignition loss and sulphuric anhydride shall not be exceeded. The use of materials other than water and calcium sulphate may be permitted when specifically called

for by the purchaser, in which case he must state the nature and the quantity of the material (s) permitted.

The remainder of the specification has to do with packaging, packing, marking, etc., and is not of special significance to this discussion, with the exception of the following note:

The hydraulic cement covered by this specification is intended for use with aggregate for making mortar and concrete for usual or normal uses. When high early strength, special resistance to solutions of sulphate or other aggressive salts, or lower heats of reaction with water are desired, use may be made of Federal Specifications Nos. SS-C-201, SS-C-211 and SS-C-206, respectively.

HIGH EARLY-STRENGTH CEMENT

The specification for High Early-Strength Portland Cement contains this notation:

The hydraulic cement covered by this specification is intended for use with aggregate in making mortar and concrete where a higher strength at early ages is desired than can be obtained by the use of cement covered by Federal Specification No. SS-C-191a for Cement; Portland.

The chemical requirements are the same as those for normal cement, except that the limit on sulphuric anhydride has been increased to 2.50 per cent. The minimum specific surface has been increased to 1900 sq. cm. per gram. The tensile strengths at 1, 3, and 7 days are required to be equal to or greater than 275, 375, and 425 p.s.i., respectively, and the compressive strengths 1250, 2500, and 3500 p.s.i. at the same ages. The other requirements are the same as for normal portland cement.

With the exception of the necessary finer grinding and increase in the allowable quantity of retarder, the only differences between the normal and the high early-strength specifications are in the strength requirements. The troublesome question of whether or not to permit additions subsequent to calcination of materials other than water and/or gypsum has been resolved by permitting any addition called for by the purchaser, provided the nature and quantity of the addition(s) are stated. The committee has taken the stand that while such additions should not be prohibited, the number and variety now being marketed make it impracticable for the committee to accept responsibility either for their effectiveness or for absence of harmful effect.

The recently adopted A. S. T. M. specification for high early-strength portland cement has attempted to solve this question by permitting additions not to exceed 1 per cent "provided such materials have been shown not to be harmful by tests prescribed and carried out by Committee C-1 on cement."

MODERATE HEAT OF HARDENING CEMENT

In the specification for "Moderate Heat of Hardening Cement," the following notation appears:

The hydraulic cement covered by this specification is intended for use with aggregates for making mortars and concretes when a lower heat of reaction (by at least 10 calories per gram at 7 days)* with water is desired than that obtainable by the use of cement furnished under Federal Specification No. SS-C-191a for Cement; Portland. The purchaser should bear in mind that such low heats of reaction are generally accompanied by lower strength at early ages.

The chemical requirements remain the same as for normal portland cement with the following exceptions:

(a) The maximum alumina content is decreased to 6.00 per cent.
(b) The tricalcium aluminate is decreased to 8.00 per cent.
(c) Maximum and minimum limits are fixed for the alumina-iron oxide ratio at 2.00 and 0.7.
(d) The minimum per cent silica is fixed at 21.00.

No limitation on silica content is included in the specification for normal and high early-strength cements.

The fineness is increased to 1800 sq. cm. per gm.

In keeping with the lower strengths at earlier ages, the tensile and compressive strength requirements are lowered as incidated in the table.

A new requirement is included under the heading "Heat of Hydration," which reads:

The heat of hydration in calories per gram shall not exceed 70 at 7 days and 80 at 28 days.

SULPHATE-RESISTING CEMENT

In the specification for the sulphate-resisting cement the following notation appears:

The hydraulic cement covered by this specification is intended for use with aggregate for making mortar and concrete when higher resistance to the disintegrative action of solutions of sulphate or other agressive salt is desired than that obtainable through the use of cement furnished under Federal Specifications Nos. SS-C-191a, and SS-C-206, Cement; Portland and Cement; Portland, Moderate Heat of Hardening, respectively.

It will be noted that the chemical limitations are even more severe than for the moderate-heat cement, the sulphuric anhydride being reduced to 1.75 per cent; the magnesia to 4.0; the alumina to 4.0 per cent; the iron oxide to 4.00 per cent and the tricalcium-aluminate to 5.00 per cent. The same limits on alumina-iron oxide ratio are retained but the percent silica is specified to be not less than 24.0. The fineness remains the same as for the moderate heat cement. As a result of the increased silica content, grinding finer than for the

*The word "by" was inadvertently omitted from the printed version.

moderate-heat cement will probably be necessary to meet the strength requirements. However, because of the greater difficulty of grinding cement high in silica, it was decided to fix the minimum fineness so as to obtain proper workability and freedom from bleeding and to permit the manufacturer to determine the additional fineness, if any, necessary to pass the strength tests. The tensile and compressive-strength requirements are lower than for the moderate-heat cement and the 3-day strength and heat of hydration tests have been omitted.

It is evident that the Committee intends this to be a "de luxe" cement of great durability.

It is significant that this specification contains no requirement for a soundness test in sodium sulphate or magnesium-sulphate solution. In early drafts such a test was included. It provided for determination of the change in length of mortar test bars in a solution of sodium sulphate. However, since available data showed that the test was not always truly indicative, the committee preferred to omit it rather than to include a requirement of such doubtful reliability.

The specification for "Methods of Sampling, Inspection and Testing" which accompanies the 4 standard cement specifications contains the usual instructions covering these features. While the committee feels that these instructions contain some minor improvements over past practice, there are no outstanding changes. The quantity of water for the tensile tests is based on the normal consistency, determined in the usual manner. The cement-water ratio for the mortar cube tests is 1.88 by weight, provided the flow lies between 100-115. When the flow is outside these limits, the amount of water must be adjusted to bring the flow within these requirements.

It is of especial interest to note the increase in the number of specification restrictions from 1904 to the present time. This is more striking when it is considered that the last 4 lines of the chart, Fig. 3, (the 1936 Federal Specifications) take the place of the first line. The increase in minimum tensile strength requirements for normal cement from 150 to 275 at 7 days and from 200 to 350 at 28 days is also notable.

FIXING CHEMICAL REQUIREMENTS

The story of the new Government specifications for portland cements would not be complete without a statement concerning the method of fixing the chemical requirements. There has been much discussion of the relative merits of defining chemical limitations by oxides and/or oxide ratios, as opposed to specifying the so-called "calculated compound compositions," i. e., C_3S, C_2S, C_3A, and C_4AF.

Attention has been invited to the following specification statement relative to calculated compounds:

> The expressing of chemical limitations by means of calculated assumed compounds does not necessarily mean that oxides are actually or entirely present as such compounds.

The Committee recognized that while there are many evidences that the properties of cement are functions of the compounds present rather than of the components, there are some striking inconsistencies and it appears highly probable that the calculated compound theory does not tell the whole story. The properties of cements of the same calculated compound composition may differ appreciably when there are differences in the methods of burning and cooling. Heat treatment of the clinker and prehydration of the cement also result in variations. The effects of the minor constituents (alkali oxides, etc.) and of the glass, which may be present in more or less significant quantities, have not been established definitely. Nevertheless, the calculated compound composition method of predicting cement properties is so strikingly accurate when the cements are made under standardized laboratory conditions that it would, in my opinion, be a mistake to neglect this useful approach to a solution of the problem of specifying cement properties by limiting chemical constituents.

Furthermore, the testimony of both manufacturers and users indicates that the calculated compound method has been a valuable aid in their work. A memorandum prepared in February 1936 by R. F. Blanks, and addressed to the Chief Engineer of the Bureau of Reclamation, contains the following statement:

> There is admittedly some inaccuracy in the present method for computing the compounds in cements from the oxide analyses. However, whether the compounds in portland cement obtain in the exact theoretical computed quantities or not is inconsequential if they can be used as *practical conversion factors for transposing the oxide percentages into workable units.* The compounds then become a convenient and simply applied criterion for specification purposes and one which can be understood and interpreted by the layman and engineer as well as by the cement chemist. On the basis of tests made by the Bureau and a study of published data, the definite conclusion has been reached that in the light of present knowledge the compound composition of cements affords the most simple and practical method of specifying chemical limits for portland cements.

In a letter received last July, the Chief Engineer of the Bureau of Reclamation states:

> . . . We again wish to stress our position that compound composition rather than oxide analysis should be used in specifications to define the requirements. . . . By recent researches, the properties of portland cements are found to correlate quite satisfactorily with their computed composition, and while the computations we now use may not be the best, they nevertheless are the best we know

at present. It is also our experience that by this system we have been able to obtain a remarkably uniform product from several manufacturers. The properties of cements are not determined by oxides but by combinations of the oxides. They are complex functions of the oxides, but comparatively simple and more direct functions of the compounds.

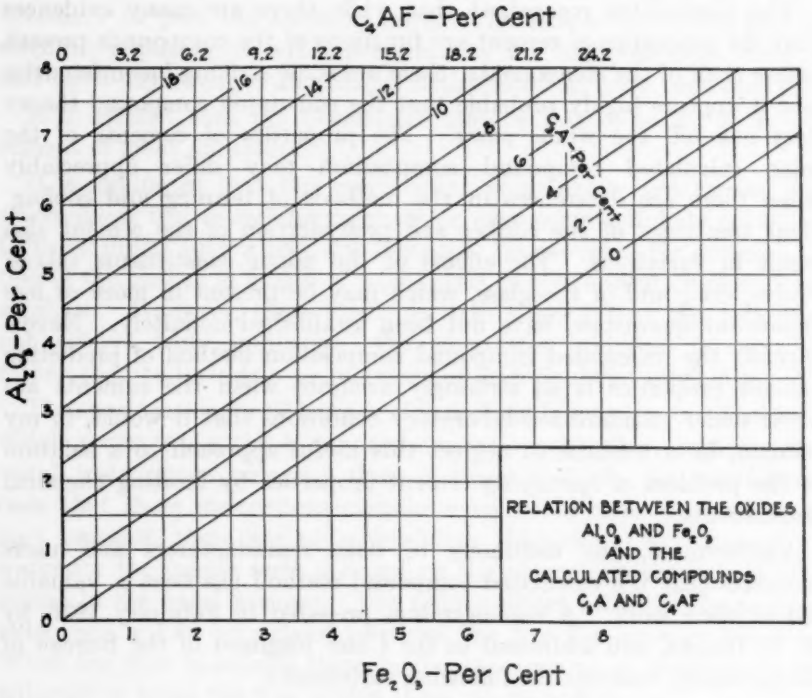


FIG. 1

While the committee did not endorse this viewpoint without qualification, it was recognized that, in principle, there is no essential difference between defining the characteristics by placing limits on the oxides and oxide ratios and by specifying the limits of the calculated compounds. In reality, the limits on calculated compounds can be transformed into oxide limits and vice versa by means of mathematical formulas. Therefore, any deficiencies in the method of specifying the qualities of cements by calculated compound compositions exist also in the oxide method.

It would appear, therefore, that the method to be adopted should be that which is easiest to use and which appeals to the practicing engineer who, in the ultimate analysis, is the one who will make most frequent use of the specification.

There is no question that the available testimony of manufacturers and users indicates that the calculated compound method has certain valuable features of which advantage should be taken. However, there are certain advantages in the simplicity of the oxide method. The committee, therefore, adopted a combination of the two methods in its specifications.

I have made reference to the interchangeability of oxide and calculated compound limitations. This can best be illustrated by reference to Fig. 1, which shows the relationship between the oxides Al_2O_3 and Fe_2O_3 and the computed compounds C_3A and C_4AF. It is apparent that any point on the chart can be fixed either by its oxide co-ordinates or by its calculated compound co-ordinates. A similar chart has been prepared to show the relationship between the lime and silica and C_2S, and C_3S.

Fig. 2 shows the limitations fixed by the new specifications for the various types of cements. The most liberal limits are established for normal and high early-strength cements. The boundaries for these two are indicated by the dotted area. The C_3A is limited to

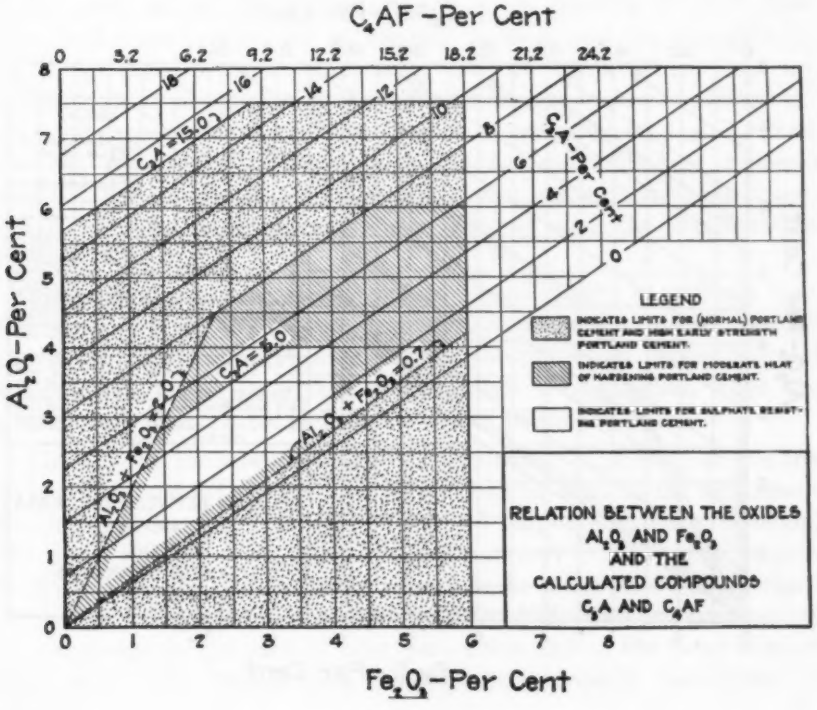


FIG. 2

15 per cent, the Al_2O_3 to 7.5 per cent and the iron oxide to 6 per cent. The cross-hatched area indicates the boundaries of the moderate-heat cement and the white area the sulphate-resistant cement.

It will be noted that the lower limit of the Al_2O_3/Fe_2O_3 ratio corresponds approximately with a C_3A content of 0 per cent to 1 per cent. This assures us a cement having no dicalcium ferrite, a compound which recent researches have indicated to be unstable and which contributes nothing to the strength.

The upper limit on the Al_2O_3/Fe_2O_3 ratio assures us of a cement with proper fluxing and setting properties. Lea and Desch, in their admirable treatise on "Chemistry of Cement and Concrete," have pointed out that "increase in the alumina with no change or with a reduction in the ferric oxide content hastens the setting of a cement and a point is eventually reached at which it becomes impossible to control the setting time adequately . . . The main role of alumina and ferric oxide in portland cement seems to be the commercially most important one of reducing the clinkering temperature and

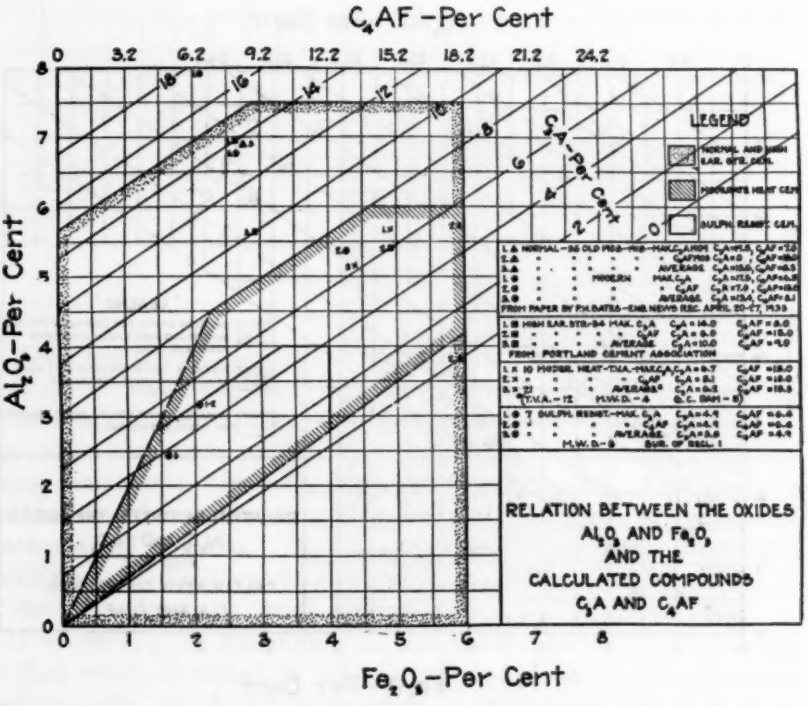


FIG. 3

rendering burning possible at temperatures which are economically attainable. In particular, a high alumina and ferric oxide content renders it easier commercially to produce a cement with such a lime content that almost all the silica can be converted into tri-calcium silicate without a content of free lime remaining in the clinker. . . . The view which is often expressed that a high alumina or alumina and ferric oxide content in a cement is favorable to the production of high early strengths is to be considered therefore as a tribute to their influence in cement burning rather than to the cementing qualities of these constituents."

It will be noted that the effect of the upper limits on alumina is to force a decrease in the C_3A content as the C_4AF increases beyond a certain amount. Beyond that amount, for each 2 per cent increase in C_4AF there must be a decrease of approximately 1 per cent in C_3A.

Fig. 3 shows points plotted for some of the cements which are or have been commercially available or which have been used on some large projects. For each class of cement one point is plotted for the cement with maximum C_3A, one point for the cement with maximum C_4AF, and one point for the average of all cements in that class. The circles are plotted for 35 modern cements and the triangles for 35 of the older cements made from 1908 to 1918. These data are taken from an article by P. H. Bates.* The squares are plotted for 34 high early-strength cements from data prepared by the Portland Cement Association. The crosses are for moderate-heat cements and the crossed circles for sulphate-resisting cements from data prepared by the Metropolitan Water District and the Reclamation Service.

The positions of the various points would appear to indicate that it is commercially practicable to manufacture cement under any of the new Federal specifications. With the exception of two normal cements, all points lie within their respective boundaries, while the averages are in all cases well inside of the boundary lines.

Fig. 4 shows the relation between the calculated C_3S and C_2S and the oxides CaO and SiO_2. The only limitations on silica in the new specifications are for the moderate-heat and sulphate-resisting cements, the limits in these cases being 21.0 per cent and 24.0 per cent, respectively. The effect of these limitations is to bring about an increase in the C_2S compound at the expense of the C_3S, which in turn results in a decrease in the rate of heat generation and in the total heat of hydration. The dotted area indicates the probable boundaries of

**Engineering News-Record*, Apr. 20 and 27, 1933.

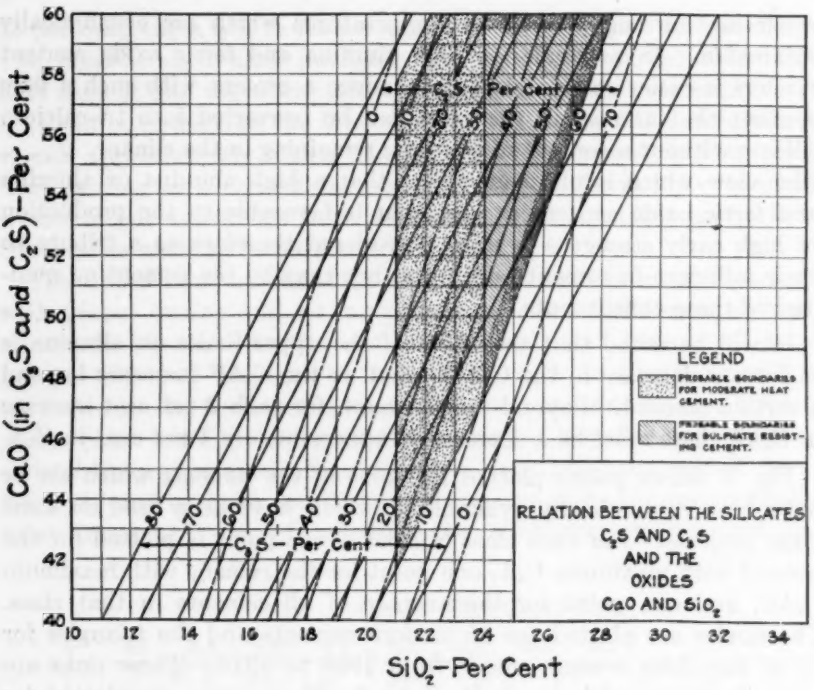

CaO (in C_3S and C_2S)-Per Cent
SiO_2-Per Cent
C_3S - Per Cent
C_2S - Per Cent
LEGEND
PROBABLE BOUNDARIES FOR MODERATE HEAT CEMENT.
PROBABLE BOUNDARIES FOR SULPHATE RESISTING CEMENT.
RELATION BETWEEN THE SILICATES C_2S AND C_3S AND THE OXIDES CaO AND SiO_2

FIG. 4

the moderate-heat cement and the cross hatched area the probable boundaries of the sulphate-resisting cement.

While the evidence as to the relative merits of C_3S and C_2S for resistance to freezing and thawing and alkali waters is not conclusive, it is generally considered that the C_2S compound is more stable in the sense that it generates less heat and shows smaller shrinkage. The performance of the two compounds when exposed to deteriorating influences would probably depend largely upon the age and degree of hydration which has been attained at the time of exposure. Tests performed by the Bureau of Reclamation indicate that higher C_2S contents are conducive to greater sulphate resistance. On the other hand, tests conducted by the Highway Research Board indicate greater resistance to freezing and thawing for cements high in C_3S.

By requiring a fairly high silica content we establish conditions which are conducive to the formation of calcium silicates, the desirable constituents of the clinker. All other things being equal, as the silica increases the undesirable diluent constituents decrease, with a consequent improvement in the quality of the cements. Also,

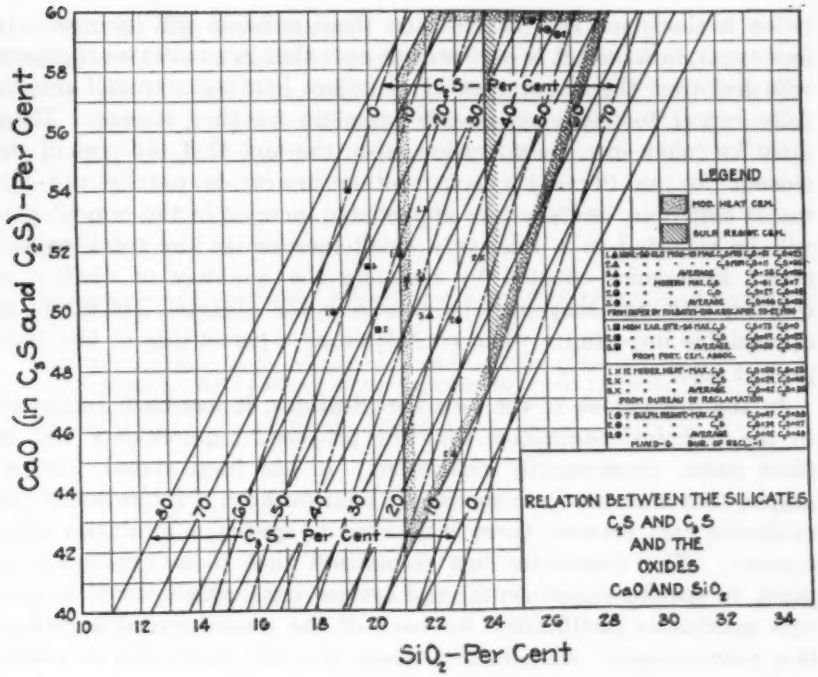

CaO (in C_3S and C_2S)–Per Cent
SiO_2–Per Cent
C_3S – Per Cent
LEGEND
MOD. HEAT CEM.
SULF. RESIST. CEM.
RELATION BETWEEN THE SILICATES C_2S AND C_3S AND THE OXIDES CaO AND SiO_2

FIG. 5

as the silica increases, there usually results a reduction in the free lime, which in turn leads to a more stable condition and, of prime importance in the case of sulphate-resisting cement, an increased durability.

In summary, I believe that while available data are not conclusive, experience indicates that by specifying a fairly high minimum silica content there are obtained cements which are better adapted to the service required of moderate heat and sulphate resisting cements.

Fig. 5 shows the same cements which were plotted on the chart for C_3A and C_4AF. Here again, there is evidence that the new specification requirements are within the range of practical commercial production.

With respect to the probable costs of these special cements, the increase in cost, if any, will depend largely upon market conditions. There are instances on record when manufacturers have been willing to make these special cements at practically no additional cost. Such instances have, however, occurred in a buyer's market. It is reasonable to suppose that as business conditions improve and compe-

tition is slackened the premium on these cements will increase. In any event, however, it is pertinent to note that in most cases engineers who feel that the importance of a project justifies a special cement fully expect to pay a reasonable premium for that cement. There must be taken into consideration, also, the fact that the cost of the cement for an important construction project is usually not the major cost item, and even an appreciable increase in the cement cost will be reflected in a relatively small increase in the total project cost. Economies effected by reducing the quantity or quality of cement, although they may be sizable in the abstract, are rarely of appreciable importance when considered as a percentage of the total project cost.

In the discussions of the new specifications, it has been suggested to the technical committee that the proposed requirements are, in some cases, unnecessarily restrictive. It has been stated, for example, that since we have specified a limit for heat of hydration of the moderate heat cement, there is no need to specify a minimum silica content. The committee has recognized that it is impossible to point to specific experimental and service data which would constitute conclusive justification for each of the recommended specification requirements. Also, in most cases it is not practicable to assure a desired quality in cement by a single specification requirement. The chemical and physical properties of portland cements are interdependent; for example, we can not establish conditions which are ideal for low heat generation without, at the same time, giving careful consideration to what is happening to the other properties, such as strength, durability, shrinkage, soundness, etc. Therefore, the committee has atempted by its specification requirements to define for each cement an area within the boundaries of which would be found the cements which have properties necessary and desirable for the intended use. It is probable that, in doing this, some cements which would show satisfactory performance in actual service will be excluded. It is believed, however, that in the present state of our knowledge, it is impracticable to avoid this with safety. It has been the object of the committee to separate the definitely good from the suspicious and the definitely bad and, at the same time, to keep within the limits of commercial practicability, i.e., we have attempted to safeguard the interests of the consumer without placing an inordinate burden on the producer. In this, I believe we have succeeded, in fair measure.

As far as I can determine, there is no thought in the mind of any member of the technical committee that the last word in cement

specifications has been spoken. These specifications are but a beginning. If the discussion incident to their formulation has done nothing else, it has demonstrated conclusively that research is the prime essential for progress in the development of portland cement. Without research, the cement industry can not survive in our highly competitive industrial society. It is hoped that cement research activities will thrive and expand to the end that we may have better specifications and better cements.*

*The opinions or assertions contained herein are private ones of the writer and are not to be construed as official or reflecting the views of the Navy Dept. or the Naval Service at large.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by June 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MARCH-APRIL 1937

Rapid and Long-Time Tests on Reinforced Concrete Knee Frames*

BY F. E. RICHART† AND T. A. OLSON‡

MEMBERS AMERICAN CONCRETE INSTITUTE

1—INTRODUCTION

Because of certain inherent advantages, the single-span rigid frame bridge has come into widespread use during the last few years. The possibilities of designing structures of this type to minimize dead load moments and to produce a shallow section at midspan were ably presented by Prof. Hardy Cross at the 1935 Convention of the Institute.[1] This one feature is of great advantage in grade separation structures, wherein the required clearance of underpass can be secured with a minimum difference in elevation of the two highway grades. Another advantage of such a bridge is that it lends itself to very attractive and pleasing exterior treatments. Among the early bridges of this type built in this country were a number having very graceful and attractive exteriors, treated in a modernistic architectural style. Probably as a result, the subsequent design of single-span frame bridges has been restricted to a rather closely standardized set of proportions, utilizing a slab with top surface conforming to the grade of the highway and lower surface curved, and walls having a vertical front face and a rear surface flared outward at the top. This architectural design has usually demanded a sharp angle of intersection between the face of the wall and the soffit of the curved slab. Although the section of the frame at the corner is normally

*Presented at the 33rd Annual Convention American Concrete Institute, New York, Feb. 23-26, 1937.

†Research Professor of Engineering Materials, University of Illinois.

‡Chicago and Northwestern Railroad Company.

[1]Hardy Cross "Why Continuous Frames," Journal Amer. Concrete Inst., Mar.-Apr. 1935, *Proceedings* Vol. 31, p. 358-67.

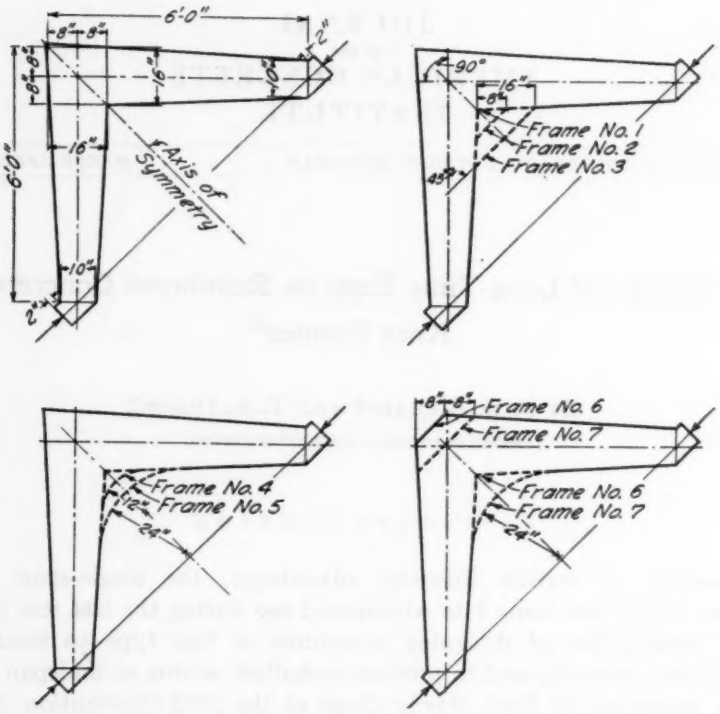


FIG. 1—ELEVATION OF KNEE FRAMES SHOWING TYPES OF FILLETS USED

made heavy as compared to the thickness at midspan, structural analysis of continuous members indicates that a reentrant angle will produce a high localization of stress. Even if a small fillet or bracket is used at the intersection, the change in direction of the members is so abrupt, and the size of the fillet so small in comparison to the depth of the section, that the stress distribution near the corner must be similar to that in a curved beam. The neutral axis of a curved beam lies between the gravity axis and the concave surface; in a member with a reentrant angle it lies near the inner surface. The stress due to a given moment is naturally much greater than in a straight beam under the same moment. This rather complicated stress situation at the knee of a rigid frame, existing as it does at the point of maximum bending moment, has generally been neglected by designers. While some experimental studies have been made which apply to the case,[2] the need for more information on the subject is evident.

[2]F. E. Richart, "Tests of the Effect of Brackets in Reinforced Concrete Rigid Frames," Research Paper No. 9, Bureau of Standards Journal of Research, 1928.

TABLE 1—OUTLINE OF TESTS OF KNEE FRAMES

Two frames of a kind. Reinforcement same in all except at fillet and in types 8-11 and 18. Rapid loading in 10 increments to failure. Sustained loading for 1 year 3 months at design stresses, with frequent deformation measurements.

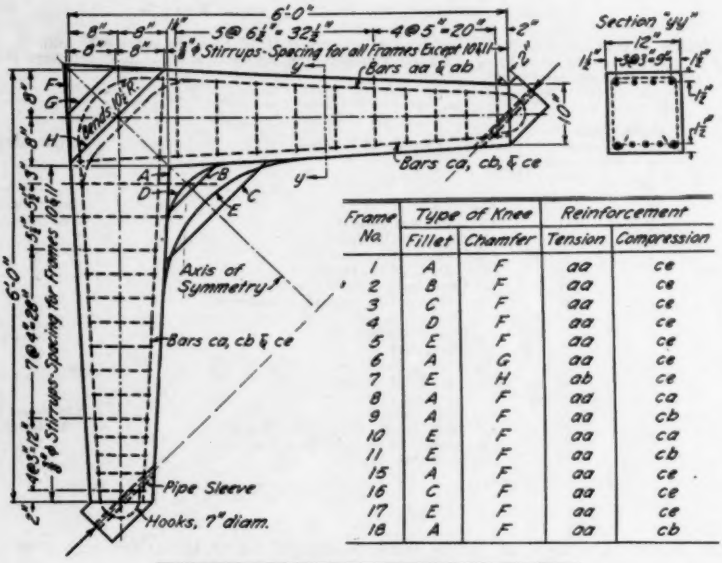
No.	Percentage of Reinforcement		Details of Fillets	Remarks
	Tens.	Comp.		
1	1	0	No fillet	Rapid loading to failure
2	1	0	45° fillet, 8-in. width	"
3	1	0	45° fillet, 16-in. width	"
4	1	0	Circ. fillet, 12-in. radius	"
5	1	0	Circ. fillet, 24-in. radius	"
6	1	0	No fillet, back corner chamfered	"
7	1	0	Circ. fillet, 24-in. radius, chamfered	"
8	1	1	No fillet. (See No. 1)	"
9	1	2	No fillet. (See No. 1)	"
10	1	1	Circ. fillet, 24-in. radius (See No. 5)	"
11	1	2	Circ. fillet, 24-in. radius (See No. 5)	"
15	1	0	No fillet. (See No. 1)	Sustained load at design stress
16	1	0	45° fillet, 16-in. width (See No. 3)	"
17	1	0	Circ. fillet, 24-in. rad. (See No. 5)	"
18	1	2	No fillet (See No. 9)	"

The present series of tests was planned to determine the moment-resisting capacity and the elastic properties of a portion of the rigid frame, using various types of fillets and arrangements of reinforcement. This includes a study of the stress distribution in the vicinity of the knee.

2—OUTLINE OF TESTS

The tests were made on special knee-frames representing a portion of the rigid frame bridge adjacent to the corner and generally subjected to negative moment The test piece consisted of a horizontal and vertical member rigidly connected and having fillets of several types at the junction, as shown in Fig. 1. The general proportions of these frames are based upon the dimensions of a full-sized rigid frame bridge, slightly modified to make the frame symmetrical about the knee. The thickness of all frames was 12 in., the overall height, about 8 ft. 9 in.

The variables studied in these tests were the size and type of fillet at the inside corner, the effect of compressive reinforcement and the effect of plastic flow. Table 1 lists the types of frame studied. The shape of the fillets used is probably most important. The fillet not only removes a high stress concentration at the corner, but it also increases the effective depth and thus strengthens and stiffens the corner section. In a rigid frame, this increase in stiffness tends to increase the negative moment and to decrease the positive moment at midspan.



Frame No.	Type of Knee		Reinforcement	
	Fillet	Chamfer	Tension	Compression
1	A	F	aa	ce
2	B	F	aa	ce
3	C	F	aa	ce
4	D	F	aa	ce
5	E	F	aa	ce
6	A	G	aa	ce
7	E	H	ab	ce
8	A	F	aa	ca
9	A	F	aa	cb
10	E	F	aa	ca
11	E	F	aa	cb
15	A	F	aa	ce
16	C	F	aa	ce
17	E	F	aa	ce
18	A	F	aa	cb

Reinforcing Bars			
Bar	No. of Bars	Diameter	Length
aa	4	¾ in.	13ft. - 7in.
ab	4	¾ in.	13ft. - 1 in.
ca	4	¾ in.	5ft. - 11in.
cb	2	¾ in.	5ft. - 11in.
	3	1 in.	5ft - 11in.
ce	2	¾ in.	5ft. - 0in.

FIG. 2—DETAILS OF REINFORCEMENT IN KNEE FRAMES

TABLE 2—AVERAGE TENSILE PROPERTIES OF REINFORCING STEEL

All bars round: ⅜-in. bars plain, others deformed.

Bar Diam. in.	No. of Tests	Yield Point p.s.i.	Ultimate p.s.i.	% Elong. in 8 in.	% Reduction in Area
⅜	21	56800	81200	18.4	55.6
¾	8	48300	72600	20.7	51.7
⅞	10	44200	73100	19.2	43.4
1	14	48700	79500	19.1	36.7

3—DETAILS OF TEST FRAMES

The frames were poured in steel forms, in an upright position, with one leg vertical, one horizontal. Concrete of proportions 1:3¼:4, by weight, a water-cement ratio of 1.2 and a slump of 4 to 7 in. were used. The materials were a standard portland cement, purchased locally, and Wabash river sand and gravel. Five 6 by

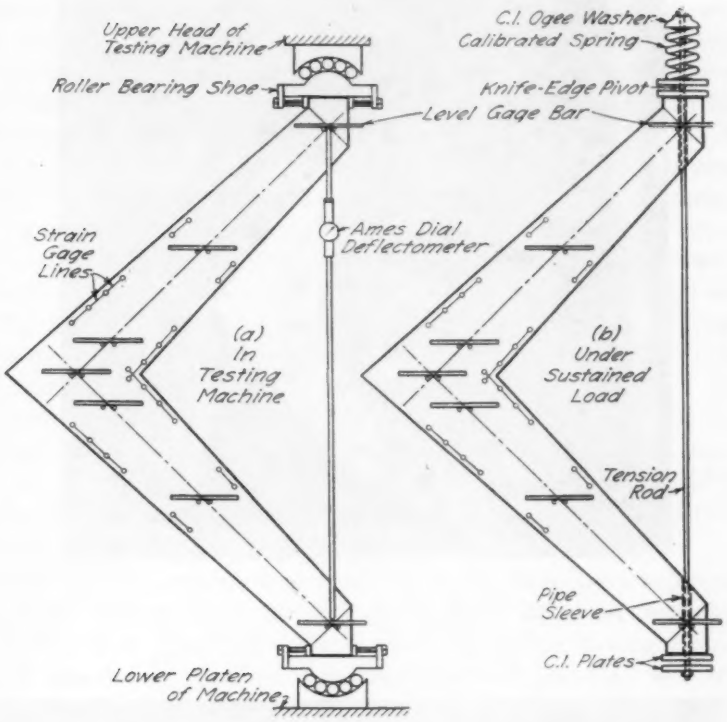


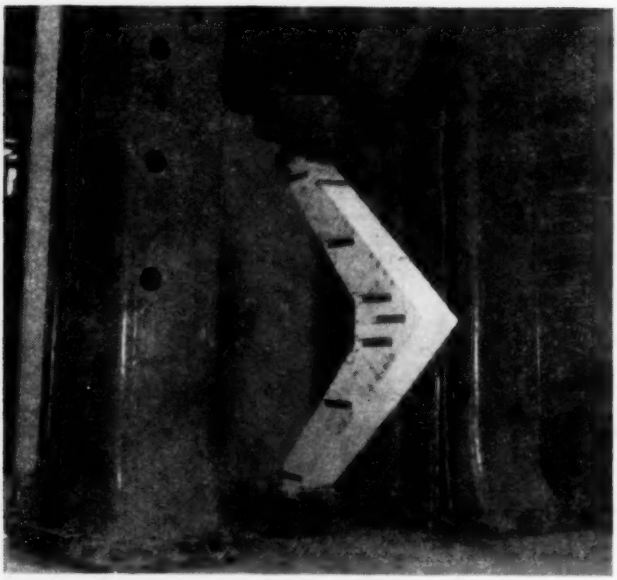
FIG. 3—ARRANGEMENT OF KNEE FRAMES FOR TESTS

12-in. control cylinders were made, stored and tested with each frame. The frames were cured under wet burlap for 28 days, and then tested.

Details of the reinforcement used in all frames are indicated in Fig. 2 and properties of the reinforcing steel are listed in Table 2. The strengths of the concrete control cylinders are given with the data of the frame tests.

4—TESTING PROCEDURE

Twenty-two knee-frames, two each of types 1 to 11, were tested to failure in a Southwark-Emery testing machine. The loads were applied through special roller bearing shoes for which the center of rotation fell at the middle of the bearing surfaces of the knee-frame, as shown in Fig. 3(a). These shoes allowed practically free rotation of the frame ends in the plane of the frame. With this arrangement it is seen that the bending moment at any point is equal to the product of the vertical load applied at the center of the bearing face and its



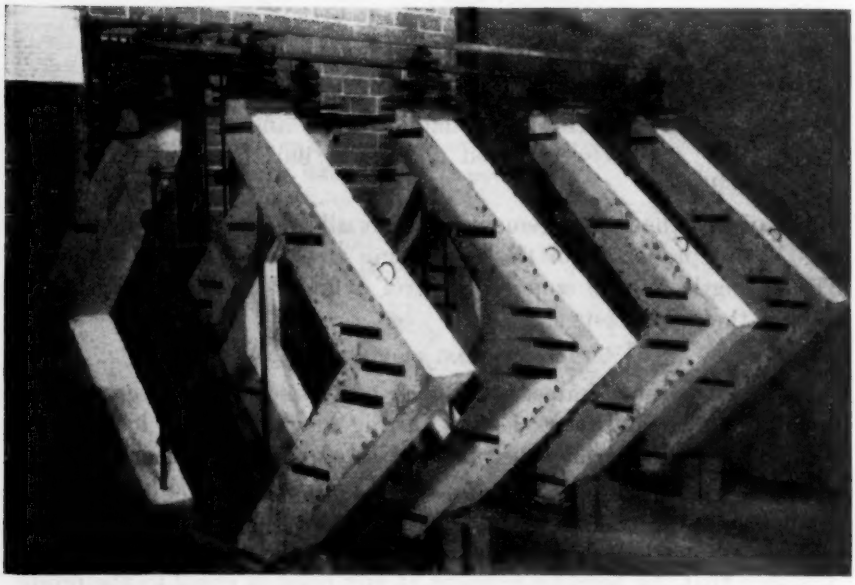


FIG. 4—TEST FRAMES UNDER RAPID AND SUSTAINED LOADING

moment arm, measured to the centroid of the frame cross-section. The moment varies from zero at the load points to a maximum at the corner. The shear and thrust at any point are equal to the components of the load normal to and parallel to the axis of the frame.

In the tests, the loads were applied to failure in about 10 increments, and strain measurements were taken at each load increment. Three types of measurements were taken; strain gage readings, on 4-in. gage lines along inside and outside faces of the frame, rotations or changes in slope at various points in the frame, and relative vertical deflections of the ends. The gage lines for these readings are shown in Fig. 3(a). The strain gage used was of the Berry type. Rotations were measured on short lengths of steel angle bolted to the sides of the frame, by means of a portable 10-in. level bar provided with a 15 second bubble and a micrometer dial graduated to 0.001 in. A similar dial was used on the vertical deflectometer used between the two ends of the frame. In the frames with fillets, some additional strain measurements were taken to study the stress distribution around the corner.

Eight frames, two each of types 15 to 18, have been subjected to sustained loading for 1 year and 3 months. The load was applied, as indicated in Fig. 3(b). A calibrated car spring having a shortening of about 1 in. at the design load was used and load was applied by tightening the nut on the tension rod connecting the extremeties of the frame until the desired load was indicated by the shortening of the spring. The load was transmitted to the frame through bearing plates and knife-edge pivots. Gage readings were taken before and after applying load, then daily for the next week and at gradually increasing intervals thereafter. The spring loads were adjusted regularly to correct for deflection due to time yield.

The frames were loaded after 28 days moist curing and stood in the air of the laboratory, in which temperature and humidity were recorded, though not closely controlled.

Views of frames being tested in the testing machine and of the group subjected to sustained loading are shown in Fig. 4 and 5.

5—RESULTS OF RAPID LOADING TESTS

Some of the principal results of the rapid loading tests of frames are given in Table 3. The average slump of the concrete used in each frame, the cylinder strength and the average initial modulus of elasticity from the cylinder tests are listed, together w th the maximum load carried by each frame. It is noteworthy that cracking began in all of the frames at a load of about 10,000 lb., and most

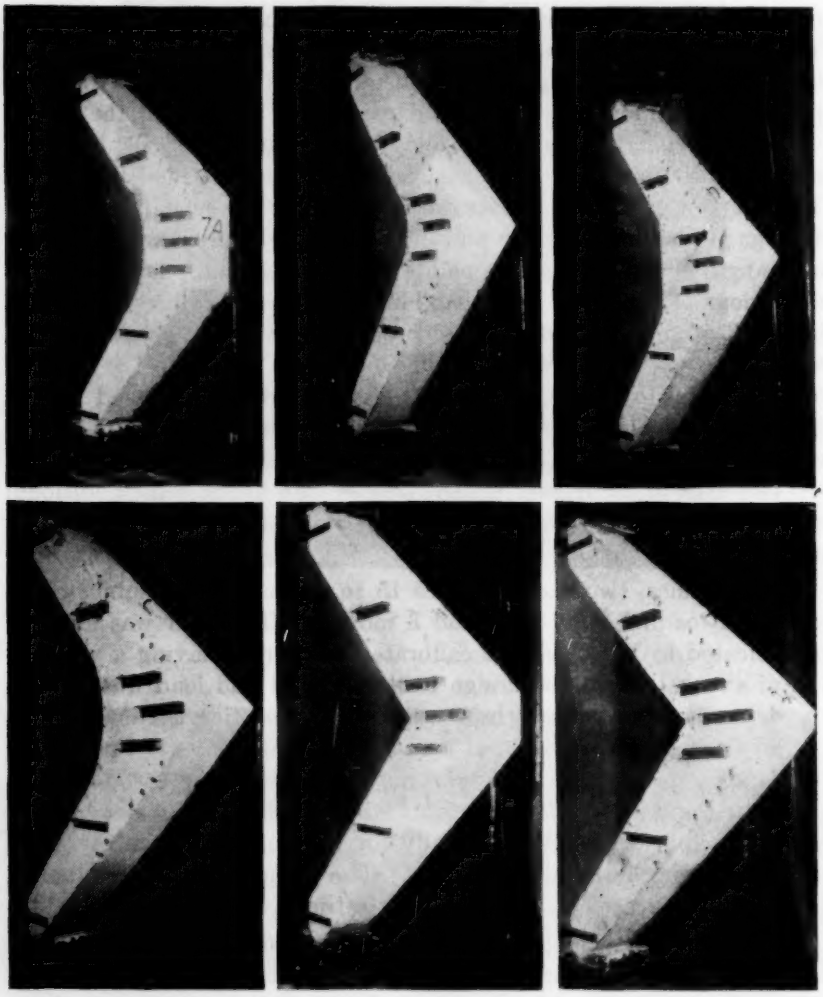


FIG. 5—FRAMES OF SEVERAL TYPES AFTER TEST

of the frames failed initially by exceeding the yield point of the tensile reinforcement. With further loading, secondary crushing occurred in a number of the frames, and in some cases it was hard to determine which was the initial cause of failure.

Comparing the average frame strengths given in Table 3, it appears that chamfering the back corner (compare No. 1 and 6, No. 5 and 7) resulted in a strength increase of about 2 per cent in each case. This

TABLE 3—PRINCIPAL RESULTS OF RAPID LOADING TESTS

Frame No.	Av. Slump, In.	Av. Cylinder Strength p.s.i.	Av. Modulus of Elasticity—Thousands of p.s.i.	Maximum Load on Kneeframe, lb.	Remarks
1-A	4.2	3660	3460	32600	No fillet
B	5.8	3450	3080	31200	
Av.	5.0	3545	3270	31900	
2-A	7.0	3480	3060	35300	45° fillet, 8″ wide
B	7.3	3440	3260	36100	
Av.	7.2	3460	3160	35700	
3-A	6.6	3270	3360	42600	45° fillet, 16″ wide
B	7.0	3350	3350	41800	
Av.	6.8	3310	3355	42200	
4-A	6.1	3300	3140	35000	Circ. fillet, 12″ rad.
B	6.3	3310	2890	35000	
Av.	6.2	3305	3015	35000	
5-A	4.1	3320	3280	41700	Circ. fillet, 24″ rad.
B	4.8	3380	3280	41600	
Av.	4.5	3350	3280	41650	
6-A	5.8	3400	3460	32600	No fillet. Corner chamfered
B	4.4	3260	3730	32300	
Av.	5.1	3330	3595	32450	
7-A	5.9	3080	3170	42800	Circ. fillet, 24″ rad. Corner chamfered
B	6.0	2760	3260	42200	
Av.	6.0	2920	3215	42500	
8-A	4.1	3010	3440	32900	No fillet. 1% comp. reinf.
B	5.5	2960	2990	32400	
Av.	4.8	2985	3215	32650	
9-A	6.0	3140	3390	33100	No fillet. 2% comp. reinf.
B	5.9	3150	3190	35600	
Av.	6.0	3145	3290	34350	
10-A	5.7	2980	3010	42000	Circ. fillet. 24″ rad. 1% comp. reinf.
B	6.3	3090	3090	43000	
Av.	6.0	3035	3050	42500	
11-A	5.8	3030	3250	42900	Circ. fillet. 24″ rad. 2% comp. reinf.
B	7.0	3080	3080	43700	
Av.	6.4	3055	3165	43300	
Gr. Av.	5.8	3220	3240		

difference is so small that it may be considered accidental. Compared to frame No. 1, the fillets of frames 2 and 4 apparently increased the strength 10 to 12 per cent, while the larger fillets of frames 3, 5 and 7 produced an increase of 31 to 33 per cent. In a similar way, comparing frames 8 and 10 and frames 9 and 11, it appears that the increase in strength produced by the large fillet was 26 to 30 per cent.

TABLE 4—MAXIMUM STRAINS NEAR ULTIMATE LOAD

Strains are given in one hundred thousandths inches per inch. Strains beyond range of gage are recorded as Y. P.

Frame No.	Per Cent of Max. Load	Max. Steel Strain	Max. Concrete Strain	Manner of Failure	Frame No.	Per Cent of Max. Load	Max. Steel Strain	Max. Concrete Strain	Manner of Failure
1A	0.90	136	115	Tension	1B	0.85	150	143	Tens. &
	1.00	331*	195			0.96	382*	215	Comp.
2A	0.93	203*	135	Tension	2B	0.93	138	228	Tension
	1.00	Y.P.*	328*			1.00	Y.P.*	391*	
3A	0.89	131	104	Tension	3B	0.91	155	141	Tension
	1.00	388*	166			1.00	253*	150	
4A	0.86	135	150	Tens. Splitting & Comp.	4B	0.86	139	154	Tens. &
	1.00	248*	323*			1.00	339*	248	comp.
5A	0.92	152	152	Tension	5B	0.92	206*	161	Tens. &
	1.00	215*	194			1.00	234*	399*	Comp.
6A	0.92	145	144	Tens. &	6B	0.93	202*	193	Tens. &
	1.00	197*	250	Comp.		0.94	Y.P.*	337*	Comp.
7A	0.89	161	180	Tens. &	7B	0.90	152	168	Tens. &
	1.00	326*	266*	Comp.		0.98	455*	208	Comp.
8A	0.91	157	144	Tension	8B	0.93	190*	170	Tension
	0.99	243*	258*			0.99	305*	213	
9A	0.91	156	76	Tension	9B	0.85	164*	86	Tension
	0.96	156	107			0.95	351*	104	
10A	0.91	164*	144	Tension	10B	0.89	135	156	Tension
	0.99	238*	185			0.99	214*	201	
11A	0.89	139	104	Tension	11B	0.87	139	144	Tension
	1.00	294*	138			0.98	403*	249	

*A steel strain of 165 or more indicates yield point of steel. Ultimate concrete flexural strains generally range from 150 upward, sometimes reaching 200 to 250.

If the frames failed initially in the tensile steel, the percentage of compression steel should have little effect upon ultimate strength. Comparing No. 1, 8, 9 and No. 5, 10, 11, it is seen that the addition of 1 and 2 per cent of compression steel resulted in slight and consistent increases in strength, amounting to 2 to 7 per cent.

The increased strength due to fillets may be due to two conditions; a reduction in the moment arm of the applied load at the point of failure and a reduction in the localized stress due to the reentrant angle at the corner. Considering the frames of Types 1 to 5, the following data are enlightening.

Frame No.	1	2	3	4	5
Relative Strength	1.00	1.12	1.33	1.10	1.31
Computed load required to produce equal values of M/jd at point of failure	1.00	1.12	1.22	1.14	1.27

The values of M/jd used in preparing this table are based upon a moment arm measured to the gravity axis of the frame and a depth

measured radially and roughly perpendicular to the gravity axis. There is some approximation involved in this procedure, but it is simple and workable.

From the above tabulation it appears that the strengthening effect of the fillets is mainly due to the reduction in moment and increased depth of member at the section near the junction of fillet and main member, where failure occurred. It appears that frames No. 3 and 5 are the only ones which show any excess of strength not predictable from ordinary strength calculations, and this excess is relatively small.

There is not much indication that any of the frames failed by localized compression, and the relative strengths tabulated above are about what would be expected from tension failures. The steel percentage at the critical section varied from 1.0 to 1.2 per cent. A weaker concrete or a larger steel percentage might demonstrate better the effectiveness of the fillets in preventing localized crushing.

To study more carefully the behavior of the frames near failure, values of the maximum measured strains are assembled in Table 4. The load increments used were from 6 to 10 per cent of the maximum load, hence they do not indicate exactly when the yield point was reached, but it is evident that it was exceeded in every case at the maximum load. Further, tension seems to be the initial cause of failure in nearly all cases.

6—ELASTIC PROPERTIES OF FRAMES

Of equal interest with the strength values are the elastic properties of the several frames. These are indicated by the measured rotations and end deflections, which were determined at about ten increments of load for each frame. For this study, values of these quantities at loads of 12,000 and 20,000 lb. have been summarized in Table 5. Gage points 1 and 7 are practically at the ends of the frame, points 2 and 6 are at the mid-length of each leg and points 3 and 5 are at a distance 8 in. each side of the intersection of the projected center lines of the legs.

Of main interest are the overall rotation and deflection between points 1 and 7. As an indication of the stiffness of the several frames, relative values of these overall rotations and deflections, derived from both loadings of Table 5, are given below. To compare with these, similar relative values have been computed from the dimensions of the frames,[3] based on the use of concrete section only, and

[3]The following equations were used. Deflection, $f = P\Sigma\frac{y^2 ds}{EI}$; rotation, $\phi = P\Sigma\frac{y ds}{EI}$ and moment of inertia, $I = \frac{bd^3}{12}$. Each frame was cut up into short lengths ds, having a depth d and a distance y from centroid to the action line of the load P.

with a constant modulus of elasticity; these values appear in the following tabulation.

Frame No.		1	2	3	4	5	6	7
Relative Test Values	Rotation	100	86	65	87	70	100	80
	Deflection	100	72	52	80	58	92	67
	Average	100	79	58	83	64	96	74
Relative Computed Values	Rotation	100	87	73	86	74	100	76
	Deflection	100	78	58	78	61	97	64
	Average	100	82	65	82	67	98	70

This simple comparison indicates a fair agreement between test and calculated values, as far as relative stiffness of frames is concerned. No attempt was made to compare absolute deflections or rotations, since the computations were applicable only to an uncracked plain concrete member. A detailed study of the measured changes in slope from point to point of the various frames with consideration of the amount of cracking, the participation of the reinforcing steel and the shape of the stress-strain diagram for the concrete will be made later.

7—RESULTS OF LONG-TIME LOADING TESTS

Eight frames, Types 15, 16, 17 and 18, were held under a sustained load of 12400 lb. for 1 year and 3 months. This load was chosen as the load which would produce a computed working stress in the steel of Frame 15; to facilitate comparisons the same load was used on the other frames. The increase in measured strains, rotations and deflections due to time yield or plastic flow appear to be fairly typical for flexural members under sustained loads. Fig. 6 shows the relation between rotations and deflections of the four frames and duration of loading. The figure shows in a striking way the stiffening effect of the fillets of frames 16 and 17 as compared to 15. Even more noteworthy is the comparison of frame 18, which had 2 per cent of compressive reinforcement, and frame 15, which had none. Not only were the initial rotations and deflections of frame 18 appreciably less than for frame 15, but the plastic yielding of frame 18 was held to practically one-half of that for frame 18. While the effects of fillets and of compressive reinforcement have a different time-characteristic, the 2 per cent of compressive reinforcement apparently stiffened the frame about as much as the fillets of frames 16 and 17.

Fig. 7 shows the relation between concrete and steel strains and the duration of loading. The relative values of concrete strains are similar to those for rotations and deflections. Compared to frame 15,

TABLE 5—ROTATIONS AND END DEFLECTIONS OF FRAMES

Rotations measured by level bar between gage points as shown in Fig. 3. End deflections measured between level bar gage points No. 1 and 7. Rotations in 1/100,000 of a radian, deflections in inches

Frame No.	At Load of 12000 Lb.				At Load of 20000 Lb.			
	Rotation between Gage Points			End Deflection	Rotation between Gage Points			End Deflection
	1–7	2–6	3–5		1–7	2–6	3–5	
1A	388	285	32	.105	910	627	128	.253
B	360	286	20	.125	910	740	72	.275
	374	286	26	.115	910	684	100	.264
2A	370	250	6	.087	830	570	7	.200
B	280	182	16	.070	720	502	3	.190
	325	216	11	.079	775	536	5	.195
3A	238	110	0	.055	560	330	0	.133
B	244	133	6	.060	610	350	3	.145
	241	122	3	.058	585	340	2	.139
4A	290	205	11	.084	775	550	15	.200
B	345	240	15	.096	825	605	5	.225
	319	223	13	.090	800	578	10	.213
5A	275	130	3	.060	604	355	0	.155
B	258	137	11	.067	625	327	5	.160
	267	134	7	.064	615	341	3	.158
6A	352	255	38	.085	850	640	150	.225
B	400	285	44	.114	960	685	134	.280
	376	270	41	.100	905	663	142	.253
7A	270	150	8	.062	702	451	25	.183
B	324	184	21	.081	760	437	24	.190
	297	167	15	.072	731	444	25	.187
8A	372	267	58	.108	845	630	190	.267
B	410	294	33	.118	920	655	82	.273
	391	281	46	.113	883	643	136	.270
9A	372	280	52	.105	830	604	140	.242
B	307	205	15	.087	755	524	30	.210
	340	243	34	.096	793	564	85	.226
10A	164	95	0	.053	550	342	3	.164
B	252	150	0	.053	630	396	8	.148
	208	123	0	.053	590	369	6	.155
11A	222	127	5	.055	555	330	2	.140
B	244	95	6	.065	595	320	4	.154
	233	111	6	.060	575	325	3	.147

the fillets of frames 16 and 17 produced a marked decrease in the maximum strains, while the strains in frame 18, with compression reinforcement, were only 45 to 55 per cent of the corresponding values for frame 15. The strains in the tensile steel, shown in Fig. 7, show very little change with duration of loading; the small variations shown probably are due to errors of observation, and to variations in temperature and humidity.

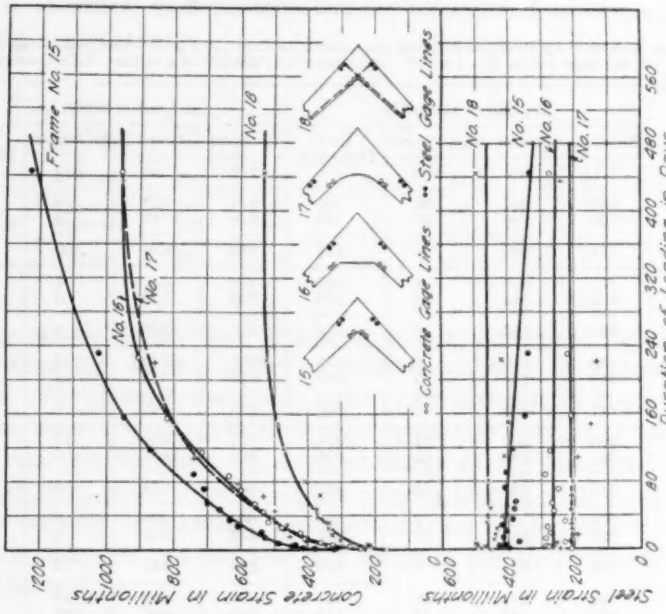


FIG. 7—TENSILE AND COMPRESSIVE STRAINS IN FRAMES UNDER SUSTAINED LOADING

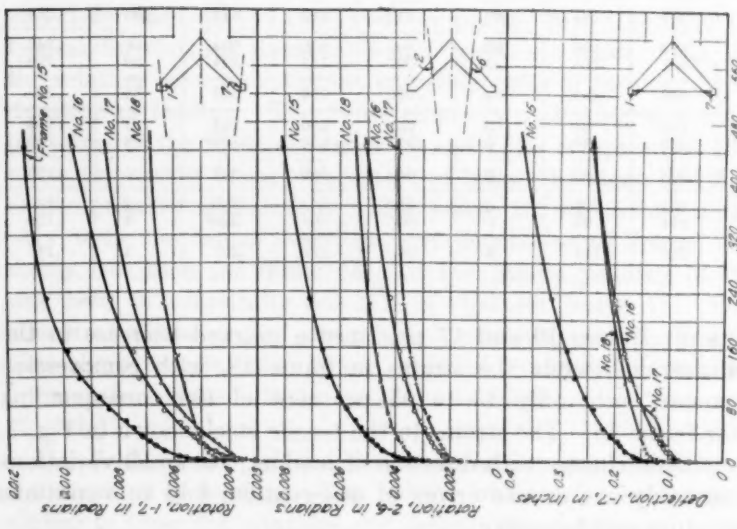


FIG. 6—ROTATIONS AND DEFLECTIONS OF FRAMES UNDER SUSTAINED LOADING

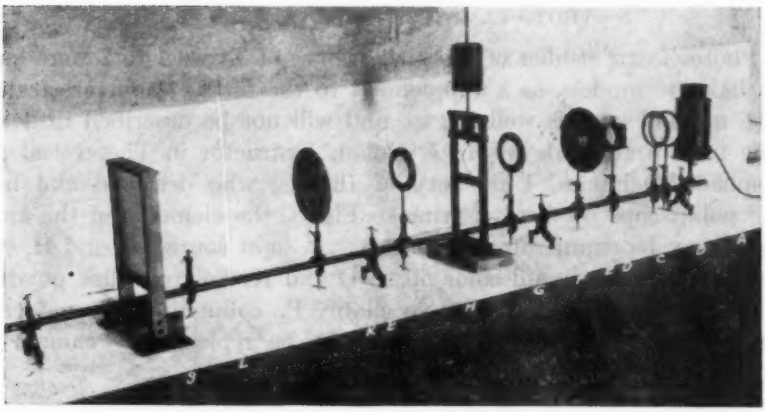


FIG. 8—VIEW OF POLARISCOPE

A summary of the time yield effects in these frames is given in Table 6, wherein the deflections, rotations and concrete strains after loading periods of 1, 6 and 15 months are expressed in terms of the corresponding initial values measured when the load was first applied. It is seen that at the end of the 15-month period, the various displacements have become 2.5 to 4.1 times their original values for frames 15, 16 and 17, and 1.8 to 2.9 times the original values for frame 18. The effect of time yield is most pronounced in the value of concrete strains.

It may be concluded that in the four types of frame tested, the use of compressive reinforcement is most effective in stiffening the frame and in reducing time yield effects, but probably adds little to the ultimate strength of the frame if it fails in tension. The filletted frames of Types 16 and 17 are much stronger and stiffer than Type 15, hence a given load produces relatively smaller deformations. The two are of nearly equal effectiveness.

TABLE 6—EFFECT OF THE YIELD OF FRAME DEFORMATIONS

All frames, No. 15 to 18, in duplicate, under sustained load of 12400 lb. Frames loaded after 28 days moist curing, and stored in air of laboratory. Strains compared are on most highly strained gage lines, average of 2 lines on 2 companion frames.

Kind of Deformation	Ratio of deformation after sustained loading to initial deformation											
	After 1 Mo.				After 6 Mo.				After 15 Mo.			
Frame	15	16	17	18	15	16	17	18	15	16	17	18
Vert. defl.	1.65	1.95	1.67	1.42	2.52	3.10	2.80	1.86	3.10	4.00	3.60	2.28
Rotation, 1-7	1.64	1.70	1.55	1.34	2.42	2.60	2.40	1.63	2.70	3.20	2.83	1.84
Rotation, 2-6	1.62	1.76	1.54	1.40	2.27	2.58	2.30	1.73	2.68	3.23	2.50	1.94
Concrete Strains	1.85	2.00	2.11	1.92	3.03	3.47	3.63	2.83	3.74	3.91	4.16	2.94

8—PHOTO-ELASTIC STUDIES OF KNEE FRAMES

Photo-elastic studies of the knee frames of Types 1 to 7 were made on bakelite models, as a supplement to the tests of concrete frames. The method used is well known and will not be described in detail. The tests were made by T. J. Dolan, Instructor in Theoretical and Applied Mechanics, University of Illinois, who designed and built the polariscope used. Referring to Fig. 8, the elements of the apparatus may be enumerated as follows: A, light source; B and H, condenser lenses; C, liquid color filter; D and K, Polaroid disk polarizer and analyzer; E, quarter-wave plate; F, collimator; G, model in loading frame; S, screen for image, to be replaced by camera for taking fringe photographs.

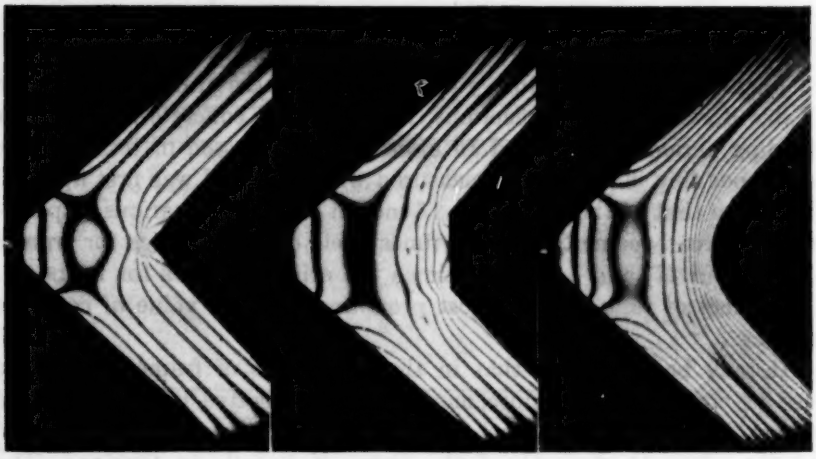


FIG. 9—FRINGE PHOTOGRAPHS OF LOADED MODELS

In the tests of bakelite models, two operations were performed. First a model was placed under the load in the field of the polariscope and the image focused sharply on the ground glass of the camera. A photograph of the interference fringes produced by circularly polarized monochromatic light was then taken. (Fig. 9.) These fringes represent loci of constant intensity of maximum shearing stress, and are given quantitative significance by comparison with a simple beam of identical material subject to pure bending. For the second operation, the camera was removed and the model subjected to plane polarized white light. The isoclinics, or loci of all points having their principal stresses acting on planes parallel to the plane of polarization, were traced on a sheet of tracing paper attached

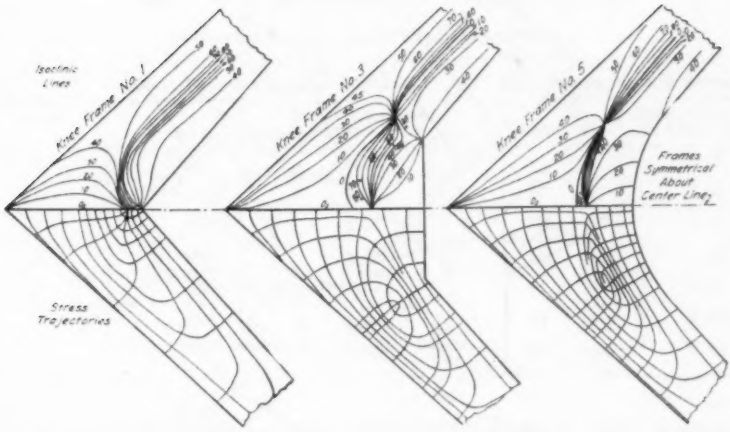


FIG. 10—ISOCLINIC LINES AND STRESS TRAJECTORIES IN BAKELITE MODELS

to the screen. Fig. 10 shows typical isoclinic diagrams, in which by rotating the polarizer and analyzer, isoclinic lines have been obtained for various angles of the plane of polarization. Thus the direction of the principal stresses may be found and "stress trajectories" plotted, as in Fig. 10.

The bakelite models used had all dimensions 1/24 of those of the reinforced concrete frames, except the thickness, which was 0.267 in. instead of 0.5 in. Since the stresses are two-dimensional, the thickness is not significant. The bakelite had a modulus of elasticity of 600,000 p.s.i., and a proportional limit of about 5000 p.s.i. While it had a tendency to creep under sustained load near this stress, it gave no trouble at the stresses of 3500 p.s.i. or less, used in the tests. Initial stresses in the bakelite as received were practically all removed by annealing it before using. The specimens were cut by sawing and filing to shape, and all models were tested within 30 minutes after they were cut out.

Having determined the difference in principal stresses from the fringe photographs, having the directions of the stresses and the location of the isoclinic lines, and knowing that one principal stress is zero at the boundaries of the member, a method of graphical integration proposed by Filon[4] was used to determine the stress at interior points of the model. The resulting principal stresses along boundaries

[4]Coker and Filon, "Photo-Elasticity," Cambridge University Press.

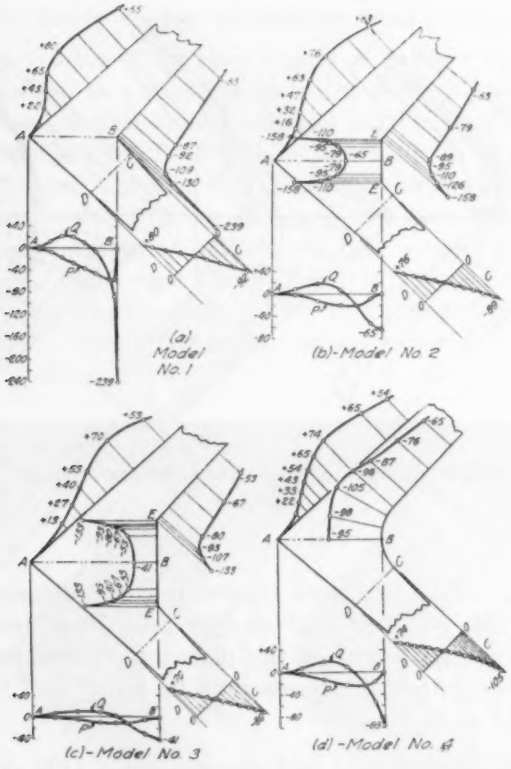


FIG. 11—STRESSES IN BAKELITE MODELS OF TYPES 1 TO 4

In p. s. i., due to 1-lb. load. Tension, +; compression, −.

and across certain sections of typical models are shown in Fig. 11. Considering the section A-B, which is the axis of symmetry of the model, the stress P represents the stress along A-B and the stress Q the stress normal to A-B. The stresses found are due to a one-pound load on the model; their significance lies mainly in their relative values at different points and in the different models. In Model No. 1, the stress Q at the inside corner is very high; at the outer corner it is zero. In model No. 2, the stress Q on the section AB is much less, and the greatest values are at the corners, E-E. In Model No. 4, the stress distribution is much smoother and the point of maximum compression is near the ends of the fillet.

Section C-D in the four models shown is approximately at the point of maximum tensile stress in the leg. The stress variation on C-D does not depart greatly from a straight-line distribution.

TABLE 7—SUMMARY OF MAXIMUM STRESSES IN MODELS

Stresses are due to a load of 1 lb. on the model.

Type of Frame	Max. Compressive Stress in Frame		Max. Tensile Stress in Frame	
	p.s.i.	Relative to Stress in Frame No. 1	p.s.i.	Relative to Stress in Frame No. 1
1	239	1.00	80	1.00
2	158	0.66	76	0.95
3	133	0.56	70	0.87
4	105	0.44	74	0.92
5	85	0.36	64	0.80
6	239	1.00	80	1.00
7	84	0.35	65	0.81

It is interesting that the point D in all of the models agrees very closely with the point of tensile failure in the reinforced concrete frames.

The maximum stresses due to a unit load as determined in models of types 1 to 7 are given in Table 7. The values of stress relative to those in model No. 1 are also given. It appears that the variation in compressive stress between models is greater, and that in tensile stress smaller, than that found in the tests of the corresponding concrete frames. The reason for the large spread in compressive stresses from the model tests is obvious; the stresses found at the inside 90° corner of frames 1 and 6 are exceptionally high and without doubt represent conditions in an elastic model. However, this high stress occurs over a very small area, it is a case of practically equal two-directional compression and the small part of the frame affected is subject to restraint in all directions. Add to this the fact that a small amount of plastic yielding would relieve this high stress concentration, and it may be concluded that the high compressive stresses found for models 1 and 6 do not indicate inversely low values of ultimate strength. The same statement may also apply to a lesser degree to models 2 and 3, in which a reentrant 135° angle results in a rather high stress concentration.

The foregoing discussion of the high stress concentrations in the models is in accord with other experience. When such localized stresses can be computed by the theory of elasticity, their values are always much higher than those indicated by tests of members in which there is a chance for plastic yielding and readjustment of the stress distribution. While the stress concentration factors from the model tests may not be applicable to concrete frames directly, the whole study gives a very good picture of the stress distribution in

knee frames. The effect of chamfering the back corner of frames 6 and 7, for example, is clearly indicated in the model tests, and the determination of the position of the neutral axis near the corner, as shown by the fringe photographs, furnishes very useful information.

9—CONCLUSION

The foregoing studies may be summarized as follows:

The tests of reinforced concrete knee frames under load applied rapidly to failure indicate clearly the increase in strength to be obtained through the use of fillets of various types. The design of the frames and quality of the materials were such that in some cases the initial cause of failure was hard to determine, but in most cases it was by tension in the reinforcing steel. In frames of weaker concrete or with higher steel percentage, the effect of high localized compression at the sharp corners would be more pronounced. Apparently, the localized compression did not produce a marked reduction in the ultimate strength of the frames with sharp interior corners.

The strengthening effect of the fillets used was largely due to the reduction in bending moment and the slightly increased depth of member at the point of failure.

The elastic properties of the frames, as reflected in the measured strains, deflections and rotations, were greatly affected by the fillets. The large fillets of frames 3 and 5 increased the overall stiffness of the frame from 50 to 70 per cent. Chamfering the back corners, as in frames 6 and 7, had very little effect on strength or stiffness.

The relative measured deflections and rotations for the various frames agreed fairly well with relative values computed by considering the gross concrete sections, neglecting the tensile reinforcement. This method of analysis for design purposes seems reasonably justified by the test results.

The tests of frames 8 to 11 show that compression reinforcement is very effective in stiffening the frames, though it added very little to the strength of the frames, because of failure in tension.

The sustained loading tests also showed the marked stiffening effects of fillets and of compressive reinforcement. After 1 year and 3 months of sustained loading, concrete strains, deflections and rotations had generally increased to 3 to 4 times the initial values, even under light working loads. The increase was least when compressive reinforcement was present.

Photo-elastic studies of scale models of bakelite showed very high compressive stress concentrations at the sharp corners of frames. These high localized stresses occur over very small areas, where the

material of the structure is highly restrained. It is likely that such stresses in the concrete frames were greatly relieved by local plastic yielding. The studies furnished data on the position of the neutral axis, the effect of chamfering corners, etc. The point of maximum tensile stress in the models agreed closely with the point of failure of the concrete frames.

The results of the tests indicate possibilities in the more effective use of material in rigid frame bridge design.

10—ACKNOWLEDGMENT

The tests described in this paper are a part of an investigation of rigid frame bridges conducted by the Engineering Experiment Station, University of Illinois, in cooperation with the Portland Cement Association. The work was under the administrative direction of Dean M. L. Enger, Director of the Station; Prof. W. C. Huntington, Head of the Department of Civil Engineering, and Prof. W. M. Wilson, Research Professor of Structural Engineering. The Association was represented by an Advisory Committee consisting of Frank T. Sheets, Consulting Engineer, and A. J. Boase, Manager of the Structural and Technical Bureau.

Especial acknowledgment is made to Paul Jones and R. H. Heitman of the Station Staff for assistance on the frame tests and to T. J. Dolan, Instructor in Theoretical and Applied Mechanics for the photoelastic tests of bakelite models.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by June 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

(Supplement)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . SEPTEMBER-OCTOBER, 1937

Discussion of a Paper by Richart and Olson:

Rapid and Long-Time Tests on Reinforced Concrete Knee-Frames*

CHARLES S. WHITNEY†

THE writer hopes that Professor Richart and Mr. Olson will extend their very valuable investigation of the action of knee frames to include tests of frames with a much higher percentage of tensile steel. Eqs. (12) and (13) cn page 495 of the March-April JOURNAL, American Concrete Institute‡ give the ultimate strength of Frame No. 1 (with $f'_c = 3545$ p.s.i. and $f_s = 48{,}300$ p.s.i.) as 32,000 pounds. The average measured strength was 31,900 lb. The frame therefore has the same strength as would be expected in a straight member under similar loading conditions and it is apparently not influenced by stress concentrations at the angle.

The same formulas (assuming the same values of f'_c and f_s) indicate that 2.83 per cent of tensile steel is required to develop the compressive strength of the concrete and that the frame would then carry an ultimate load of 67,600 pounds unless there is some weakness due to the angle which does not exist in a straight member. A test of such a frame would be of great value in indicating the effect of the stress concentrations. The fact that concrete compression failure appeared imminent in the test frames with only one per cent of steel does not mean that much greater strength could not be developed on the compression side if the tensile steel is greater because in the latter case the compression is developed over a greater area of the concrete.

The knee without fillet often has considerable advantage in appearance and in providing better clearance. The tests indicate that the

*JOURNAL Amer. Concrete Inst., Mar.-Apr. 1937; *Proceedings* Vol. 33, p. 459.

†Consulting Engineer, Milwaukee.

‡"Design of Reinforced Concrete Members Under Flexure or Combined Flexure and Direct Compression," by Charles S. Whitney.

strength is increased by fillets only because the critical sections is moved to a location of lower bending moment.

It is important to know if that holds true with higher percentages of tensile steel.

The additional stiffness provided by the fillets is not entirely beneficial because in an elastic frame construction it has the effect of increasing stresses due to temperature changes and shrinkage, and it has the further effect of increasing the bending moment at the knee by shifting the point of inflection away from it. These points are mentioned merely to discourage the over-enthusiastic use of fillets without full consideration of their effect.

HOMER M. HADLEY*

To a certain extent with respect to failure under loading and to a major extent with respect to plastic flow, the character of the concrete used in these tests appears to have influenced the results.

It is stated to be a mixture 1:3¼:4 by weight, with a water-cement ratio of 1:1.2. This 9 gal. of water per sack of cement is an unusually large quantity. Likewise, the quantity of sand in the mixture is considerable. It seems probable in the case of the legs cast in a vertical position that water gain occurred in their upper portions and that the strength of the concrete there was less than that of the companion cylinders.

Otherwise, it is difficult to see how there could be any question whatsoever as to the nature of the failure occurring with only 1 per cent tension reinforcement. Table 4 reports specimens 4A, 6A and 7A failing by "tension and compression," the other specimens failing by tension only.

In any ordinary case, it is impossible to produce a flexural compressive failure in concrete with only 1 per cent tensile reinforcement. Apparently 3 per cent or more is required. The author's Bulletin 237 of the University of Illinois' Engineering Experiment Station reports failures by tension in the steel of beams having 2.8 per cent reinforcement. The McCullough tests (*Engineering News-Record*, September 19, 1935) showed tension failures with 3.0 per cent reinforcement. Similarly with other tests. Hence the suggestion of compressive failure in the concrete here appears anomalous.

From time to time there are to be encountered engineers who are gravely troubled by "plastic flow" and "time yield" in concrete. There is doubt in their minds about its exact nature and about what it amounts to, but nevertheless it hath a sinister sound. Against these apprehensions, buildings and bridges stand without showing

*Regional Structural Engineer, Portland Cement Assn., Seattle, Wash.

deflections appreciably increasing with time. The present paper submits some definite quantitative figures on the magnitude of this property in the case of the specimens subjected to sustained loads.

It is therefore important to have attention directed to two things: (1) To the aforementioned character of concrete used in these specimens; (2) To the fact that the specimens were subjected to load at the age of 28 days, having been kept in a moist condition continuously for this time and that while under load they dried out to room conditions of temperature and humidity.

These facts go far towards explaining the reported deflections. The nature of the concrete was such as to have pronounced shrinkage on drying out; the tests began when there had been no preliminary loss of moisture from the concrete; it was while under load that the specimens dried out. It is interesting to note in Figures 6 and 7 that after 240 days practically no change in deformation of either steel or concrete was measured in frames 16, 17 and 18 at the points of maximum stress where apparently equilibrium had been established. Subsequent increased deflection in the frames must therefore have resulted from deformations elsewhere along the legs.

At the beginning of the paper it is stated that designers have been inclined to "neglect the rather complicated stress situation at the knee of a rigid frame." In view of the confinement and restraint of the material at such points and in view of the demonstration afforded by these tests, it appears that the designers may well continue in their former ways.

AUTHOR'S CLOSURE

Replying first to Mr. Hadley's discussion, the writer does not feel that the concrete mixture used in these tests was at all unusual or that it should produce any abnormality of either water gain or plastic flow. The proportions were the same as those used for 3500-lb. concrete in the A. C. I. column investigation in 1930. The high ratio of fine to coarse aggregate was necessitated by the fact that though the gravel used was a 1-in. material, most of it passed the ¾-in. sieve. The water ratio of 1.2 was used to give the consistency and strength desired; the slump of 4 to 7 in. was none too great to insure proper placing in these small reinforced members. The torpedo sand was a little finer than we have usually used. Had Mr. Hadley known these details of the mixture he probably would not have assumed that there would be undesirable water gain. The writer watched the pouring of many of these frames and there was no water gain of any significance. The frames were cast in one operation, with one leg vertical, and had

there been water gain it would have been manifest at the top of the horizontal leg, not at the inside corner of the frame.

Mr. Hadley's discussion implies that compressive stress conditions at a sharp re-entrant angle are the same as in a straight beam, since he states that from beam tests it is well known that roughly 3 per cent of reinforcement is required to produce a compression failure and that "the suggestion of compressive failure in the concrete here appears anomalous." The theory of elasticity, however, does indicate that the stress at a sharp corner is much greater than that due to the same moment in a straight beam. The photo-elastic analysis summarized in Table 7 of the paper indicates that the compressive stress at the sharp corner of Frame 1 is nearly 3 times that in the filleted frames of Types 5 and 7. That compression failures did not occur in specimens with 1 per cent of tensile reinforcement is due, in the writer's opinion, to the ability of the highly stressed concrete to yield plastically and to distribute the compressive stress, rather than to the effect of restraint at the corner. It is true that there are compressive stresses in two directions at the corner, but there is very little restraint in the third direction (normal to the plane of the frame). Tests have shown very high strength in concrete under compressive stress in three rectangular directions, but very little difference between the strengths in one-way and two-way compression.

The studies of plastic flow given in the paper were made to furnish better knowledge of the effect and limitations of this particular property of concrete. No one can object to the development of more complete information regarding the action of a material or a structure. We may well recognize that the ability of concrete to yield plastically is in many ways beneficial to reinforced concrete structures.

Mr. Whitney's suggestion that additional tests be made on frames with a higher reinforcement ratio was a good one. In the original series, the percentages were limited to those which would commonly be met in design; to see where compression failure would occur, higher percentages were evidently needed. As an indication, and not as a complete answer to the problem, two additional knee frames have been made and tested. These frames were identical in overall dimensions with Frame No 1, with the sharp inside corner, but had eleven $\frac{3}{4}$-in. bars instead of four as tensile reinforcement. These bars were placed in two layers, six in the outer layer and five in the inner layer. This reduced the effective depth slightly and increased the steel percentage from about 1.0 per cent to about 3.0 per cent. The bars were from the same lot as used in the original series of tests.

The two frames, 19A and 19B, were made as nearly like the original series as possible, but due to the use of a new lot of materials, the concrete strength was somewhat higher, averaging 4185 lb. per sq. in. Frame 19A when loaded in the testing machine gave indications of initial compressive failure at a load of 44000 lb., cracks developed parallel to the compression face and failure finally occurred by crushing of the compression face about 10 inches from the corner, at a load of 54,000 lb. Frame 19B, failed in much the same way, with initial evidence of compression failure at the load of 47,000 lb.; final crushing took place at and near the inside corner at a load of 60,250 lb. In both frames the tension cracks remained fairly small and were well distributed. There was no doubt that both were compression failures.

It may be useful to compare the results of these tests with those of Frames 1 and 6, which had 1 per cent reinforcement and no fillets. The average strength of Frames 1 and 6 was 32,175 lb., and the corresponding concrete cylinder strength was 3437 lb. per sq. in. The average strength of Frames 19A and 19B was 57,125 lb., and the cylinder strength was 4185 p.s.i. Since the concrete was definitely stronger for Frames 19A and B, it seems reasonable to adjust the ultimate load for those frames in the ratio of 3437 : 4185 in order to compare with Frames 1 and 6. The adjusted strength of Frames 19A and B thus becomes 46,900 lb. as compared with 32,175 for Frames 1 and 6. The ratio, $\frac{46{,}900}{32{,}175}$, is 1.46. This rather rough comparison indicates that the frames with 3 per cent reinforcement were a little less than $1\frac{1}{2}$ times as strong as those with 1 per cent reinforcement. It should be remembered, too, that increasing the percentage from 1 to 3 per cent lowered the neutral axis greatly and consequently increased the compressive resisting moment (as indicated by the quantity $kj\ bd^2$) by about 20 per cent. From these computations it appears that the use of 1.20 to 1.25 per cent reinforcement in the original frames of Types 1 and 6 would have resulted in a condition where either a tensile or compressive failure might have occurred. From this it is obvious that the use of 1.5 per cent of tensile reinforcement instead of the 1.00 per cent used would probably have resulted in compressive failure at the inside corner.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MARCH-APRIL 1937

Design of Reinforced Concrete Members Under Flexure or Combined Flexure and Direct Compression*

By Charles S. Whitney†
MEMBER AMERICAN CONCRETE INSTITUTE

It is the purpose of this paper to suggest a complete revision of the method of designing reinforced concrete members subjected to bending and to present a rational method for the proportioning of arch ribs, rectangular columns under eccentric load, and rectangular beams. This method may be extended to cover the design of T beams, round columns and any other form of concrete members. Simple formulas are given which predict the ultimate strength with remarkable accuracy based on the cylinder strength of the concrete and the yield point of the steel independent of the ratio of their moduli of elasticity.

It is believed that these equations will greatly simplify the work of the designer and result in the more efficient and economical use of reinforced concrete. The suggestions are intended to place structural concrete design on a solid practical foundation which is now justified by the advance in construction practices, the availability of better materials, and the information gained by a great amount of research work.

The present method of designing members under bending and combined bending and direct stress is unsatisfactory for several reasons.

The usual formulas based on assumption of a cracked section and straight line variation of stress are far from correct both under working loads when the concrete is not materially cracked and under ultimate loads when the stress in the concrete is not even approximately proportional to the distance from the neutral axis. They

*Received by the Institute Feb. 11, 1937.
†Consulting Engineer, Milwaukee.

do not give a good indication of the conditions under working loads and cannot be used to predict the ultimate strength with any degree of accuracy.

They do not properly recognize the strength of the concrete in compression, and provide a much higher factor of safety against failure of the concrete than of the steel.[2][6] That may have been justified during the early days of reinforced concrete on the grounds of unreliability but it is certainly not justified today. Properly balanced design will permit the use of a considerably higher percentage of tensile steel with a consequent reduction in the size and weight of members.

The usual flexure formulas are complicated by the use of the value of n which is quite unpredictable under high loads and actually has little effect on the ultimate strength of the beam.[2][9][10]

The effective value of n is widely different for dead and live loads. In the case of arch ribs, the dead and live load stresses cannot be computed separately with different values of n and added together, because dead load may produce compression only, while the live load moment alone would require assumption of a cracked section. The calculation in this case would be very complicated with two values of n.

The present method makes no prediction of the loading which will cause cracking and does not give accurate control over the factors of safety under dead loads and live loads.

The method here proposed for the design of members under bending and direct stress, particularly for arches, is direct and simple. First, an analysis of tensile stress in the arch rib should be made assuming the critical combination of live and dead loads, together with temperature, shrinkage and plastic flow effects. This can best be done by using the complete transformed section without cracks. The effective values of n for dead and live loads can be predicted with sufficient accuracy for this purpose at allowable working stresses and the dead and live load stresses for corresponding values of n can be computed separately and added together. The tensile stress in the concrete so determined should not exceed the modulus of rupture of the concrete in an unreinforced beam in order that the arch rib shall not be cracked in service. The compressive stress for this condition is relatively unimportant because the strength of the rib will be controlled by an ultimate strength calculation.

In arch ribs whose center lines follow the dead load pressure lines, the tensile stress can easily be kept somewhat below the modulus of rupture. In the rigid frame type of arch, it may be possible to

permit the tensile stress for the extreme case to slightly exceed the 28-day modulus of rupture without introducing objectionable cracking; or if the critical combination of loads includes only a portion of the full live load, the calculated tensile stress can probably be kept below the modulus of rupture without difficulty. This method of design would have the advantage of directing attention to the tensile stresses and would encourage the use of concrete with higher tensile strength.

This calculation using the full strength of the uncracked section might result in the use of too little reinforcing steel and provide an inadequate factor of safety unless an additional calculation is made of the ultimate strength of the rib. Because of the large strains occurring before failure, the effects of temperature change, shrinkage, and plastic flow have no practical effect on the ultimate strength.[1][3][4] It is therefore proposed that these effects be neglected and that the ultimate strength of the member be computed by a new type of formula based on the ultimate strength of the concrete, and on the yield point strength of the steel.

It has been pointed out recently by several investigators[4][9][10] that while the stress variation in the concrete is approximately linear under very light loads, and parabolic under intermediate loads, as the ultimate load is approached it assumes a shape about as shown in Fig. 1.

The stress increases very rapidly near the neutral axis and is nearly uniform for the greater part of the depth of the compression section, probably decreasing slightly toward the edge of the beam. Saliger[10] reports the ultimate strain in the concrete at the outer edge of the beam to be from .003 to .007 while the limit reached by concrete prisms at failure was .002 to .004. The usual "parabolic" formulas have been based on the theory that the stress variation in the beam

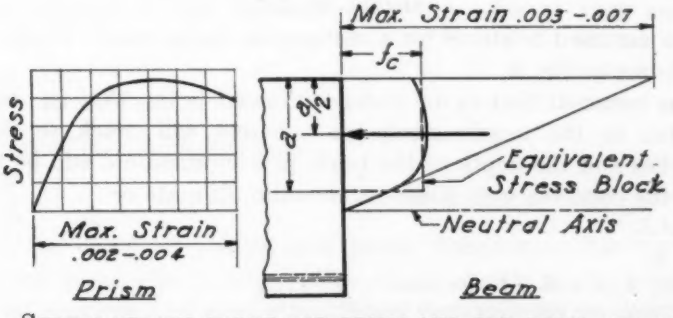


FIG. 1—STRESS-STRAIN CURVE FOR PRISM AND FOR BEAM AT ULTIMATE LOAD

followed the shape of the first part of the stress-strain curve for the concrete cylinder up to the point of maximum load.

This is evidently considerably in error because of the greater ultimate strains and the different behavior of the concrete in the beam. On this account, the writer questions the value of any refinements of beam theory based on an attempt to estimate the ultimate stresses in a beam by comparison with the stress-strain curve for the cylinder or other standards.

It is therefore proposed that a rectangular block of uniform stress as indicated by the dotted lines on Fig. 1 be used to represent whatever stress may exist in the concrete. Whatever it actually is, it must have an average intensity, f_c, and an effective depth, a. The resultant is assumed at the middle of the rectangle. Under ultimate load, Hooke's Law and the theory of elasticity have no significance as far as the internal stresses are concerned. The materials are more nearly in a plastic state but the imperfect and variable action of concrete makes a rigid solution according to the theory of plasticity impractical. No further theoretical justification of the assumption a rectangular compressive stress block is necessary if the formulas derived therefrom accurately predict the ultimate strength of the member. This they appear to do.

This assumption of uniform compressive stress on the concrete in a beam has been suggested by Gebauer and by Copée but their formulas do not appear entirely satisfactory because of their other assumptions. Since the development of the following treatment by the writer, von Emperger[9] and Saliger[10]; in excellent discussions of this subject have recommended the use of simplified ultimate strength formulas for beams independent of n. Their formulas are somewhat less simple and do not appear to check the results of American beam tests quite as well as those presented herein.

SIMPLE FLEXURE

The assumed relations for a rectangular beam under simple flexure are shown in Fig. 2.

It is assumed that in an under-reinforced beam, that is, one which will fail in the tensile steel, the concrete will crack as the steel stretches and the depth of the beam in compression a will be reduced until the concrete unit stress reaches the ultimate or

$$a = \frac{A_s f_s}{b f_c} \quad \text{Eq. (1)}$$

in which, A_s = area of tensile steel
f_s = yield point stress in steel
b = width of beam
f_c = ultimate strength of concrete

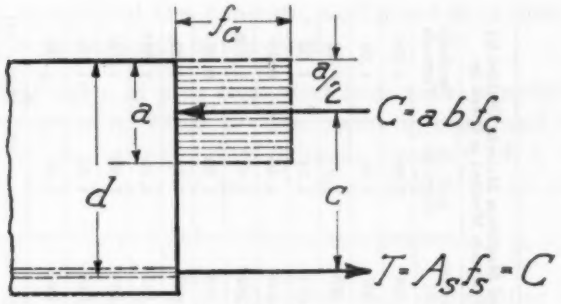


FIG. 2

This determines the lever arm of the steel reinforcement since

$$c = d - \frac{a}{2}$$

It will be assumed that the ultimate compressive strength of the concrete in the beam is equal to 85 per cent of the cylinder strength in order to be consistent with the results of tests on concentrically loaded columns and to have a treatment which can be applied to the full range of cases from simple flexure to direct load.

The values of a and c are derived as follows for any particular bending moment:

$$M = \left(d - \frac{a}{2}\right) abf_c \quad \text{Eq. (2)}$$

from which

$$a = d - \sqrt{d^2 - \frac{2M}{bf_c}}, \text{ or, } \frac{a}{d} = 1 - \sqrt{1 - \frac{2M}{bd^2 f_c}} \quad \text{Eq. (3)}$$

and

$$c = d - \frac{a}{2} = \frac{1}{2}\left(d + \sqrt{d^2 - \frac{2M}{bf_c}}\right) \quad \text{Eq. (4)}$$

These expressions are independent of the area of steel and the value of $\frac{E_s}{E_c}$.

The required steel area is simply,

$$A_s = \frac{M}{cf_s} \quad \text{Eq. (5)}$$

Since the assumed compressive stress distribution has no exact theoretical basis, the limiting value of the depth of compression a for equal concrete and steel strengths in flexure must be determined experimentally. If the beam has at least sufficient steel to fully

TABLE 1—BEAM TESTS REPORTED BY SLATER AND LYSE

Group No.	d (in.)	b (in.)	p	Cylinder strength (lbs. per sq. in.)	Maximum Moment (in lbs.)	Value of a from Eq (3)	$\frac{a}{d}$	$\frac{c}{d}$	Maximum Moment from Eq (6)	Actual Maximum Moment Divided by Maximum Moment as Given by		
										Equation (6)	A. C. I. Formula	Parabolic Formula
1	10.2	8.2	.021	1390	597,000	$>d$	>1.0*		396,000	1.508*	2.06*	1.50*
2	10.3	8.2	.028	2790	818,000	5.6	0.544	.728	811,000	1.007	1.51	1.03
3	10.3	8.2	.037	4070	1,152,000	5.3	0.515	.742	1,183,000	0.973	1.50	0.98
4	10.1	8.2	.047	4800	1,360,000	5.6	0.554	.723	1,342,000	1.013	1.54	1.00
5	10.2	8.3	.056	5740	1,584,000	4.9	0.481	.759	1,660,000	0.955	1.46	0.95
6	14.2	8.2	.030	2590	1,805,000	12.9	0.910*	.545*	1,430,000	1.262*	1.85*	1.27*
6A	14.1	8.2	.039	4130	2,196,000	7.3	0.518	.741	2,250,000	0.976	1.50	0.97
7	12.2	8.3	.028	2950	1,240,000	6.75	0.553	.723	1,220,000	1.016	1.56	1.03
8	8.0	8.1	.031	2760	543,000	5.36	0.670*	.665*	478,000	1.135*	1.66*	1.13*
9	5.9	7.9	.032	2900	310,000	4.17	0.708*	.646*	266,000	1.166*	1.69*	1.14*
10	4.1	8.0	.030	2820	129,000	2.26	0.552	.724	127,000	1.016	1.48	1.00
10A	4.1	8.0	.040	3810	179,000	2.37	0.576	.712	171,000	1.046	1.55	1.03
Average							$.537 \pm .024$	$.732 \pm .012$		$1.00 \pm .024$	$1.51 \pm .028$	$1.00 \pm .024$

*These values not included in average.

develop the strength of the concrete, additional steel does not materially increase the strength of the beam.

The limiting value of a as computed from tests reported by Slater and Lyse[2] is given in Table 1. The value of f_c assumed in Eq.(3) is 85 per cent of the corresponding cylinder strength, of, $f_c = 0.85\ fc'$. Eliminating four groups of tests which appear to be erratic, the average value of $\frac{a}{d}$ is 0.537. This is independent of p, d, and f_c.

The flexural strength of a fully reinforced rectangular beam with tensile steel only is then, from Eq.(2).

$$M = (d - 0.2685\ d)\ 0.537\ d\ b\ f_c$$
$$= 0.393\ b\ d^2\ f_c$$

$$\text{or } \frac{M}{bd^2} = 0.393\ f_c = \frac{f_c'}{3} \quad \text{.....Eq. (6)}$$

in which f_c is 85 per cent of the compressive strength of standard cylinders, f_c'. Column 10 of Table 1 gives the maximum moment for the test beams as predicted by Eq.(6). It may be noted that the percentage of error is practically the same for Eq.(6) as for the parabolic formula given by Slater and Lyse using the measured value of n.

Fig. 3 shows the value of $\frac{M}{bd^2f_c'}$ given by the 36 beams tested by Slater and Lyse and by 33 beams tested by Humphrey[11] which had cylinder strengths between 1000 and 2500 p.s.i. The Humphrey beams with higher strength concrete were not included because it appears that there was not sufficient reinforcement to fully develop such higher concrete strength even though they may have been reported as compression failures. Some confusion no doubt exists because it is difficult to differentiate between primary steel or concrete failure unless the amount of steel is considerably below the critical percentage required to develop the full concrete strength. A slight stretching of the steel may reduce the concrete compression area and cause what looks like a concrete failure in an under-reinforced beam.

The value of $\frac{M}{bd^2f_c'}$ shown by Fig. 3 is unaffected by the concrete strength between 3000 and 6000 p.s.i. although it rises about 50 per cent for weaker concretes from 3000 to 1000 p.s.i. With the present methods and materials, it is not probable that much structural con-

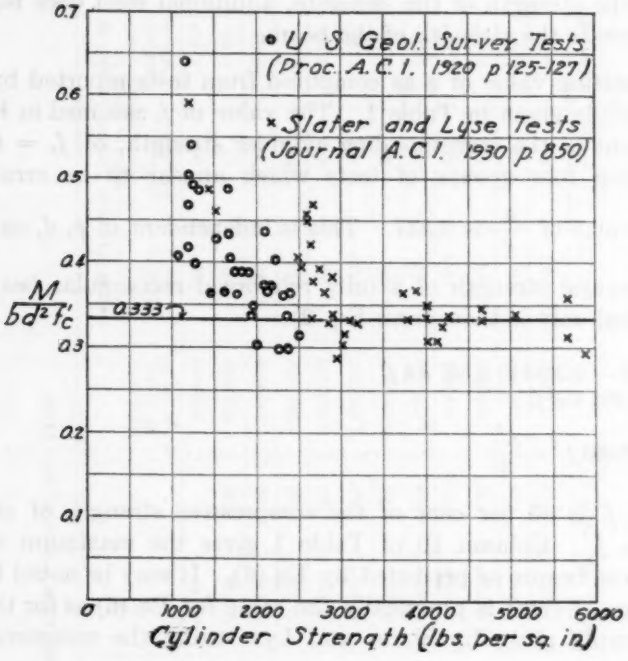


FIG. 3—VALUE OF $\frac{M}{bd^2f_c'}$ FROM TESTS OF BEAMS

crete will be used with a strength of less than 2500 or 3000 p.s.i., and if it is used, a greater factor of safety is not inappropriate. Therefore it appears that a value of 0.333 for $\frac{M}{bd^2f_c'}$ can be used generally.

For beams with less steel than required to develop the full compressive strength the allowable bending moment is given by the formula:

$$\frac{M}{bd^2} = pf_s\left(1 - \frac{pf_s}{1.7f_c'}\right) \quad \text{Eq. (7)}$$

The critical percentage of steel required to develop the full compressive strength of the concrete is:

$$p_s = 0.456\frac{f_c'}{f_s} \quad \text{Eq. (8)}$$

Table 2 shows the results of the application of Eq. (5) to 72 beams of the series tested by Humphrey[6] except beams 489-490-491 where Eq. (6) controls on account of the low strength concrete. The average

ratio of actual maximum moment to that given by the formulas is 1.071 to 1, and, eliminating the low strength cinder concrete beams, the ratio is 106.24 to 1. Classified according to age, the ratio is 1.052 to 1 at 4 weeks and 1.09 to 1 at 52 weeks. According to percentage of steel, it is 111.7 for $p = 0.0049$, 105.44 for $p = 0.0098$, and 104.02 for $p = 0.0196$. There appears to be no correlation between modulus of elasticity of the concrete and the accuracy of the formula.

The value of the steel lever arm, $\frac{c}{d}$, from Eq.(4) is given in Table 2 for comparison with the standard value, j. The value given by Eq.(4) is greater and appears to be more satisfactory than j by the usual method.

Equations (5) (6) and (7) appear to check the results of well controlled tests within the limits of the variability of the materials. They appear to be as accurate as any treatment which can be devised from information available although more complete tests may make possible a slight improvement in the empirical constant.

The effect of these formulas is shown in Fig. 4 based on 3000 p.s.i. concrete cylinder strength and 50,000 p.s.i. steel yield point with a factor of safety of 2½ for both concrete and steel.. The allowable value of $\frac{M}{bd^2}$ is plotted against the percentage of steel and compared with the value by the usual formula with $f_c = 1200$ p.s.i., $f_s = 20{,}000$ p.s.i. and $n = 10$. The new formulas raise the critical percentage of steel from 0.0113 to 0.0273 and the allowable $\frac{M}{bd^2}$ from 197.5 to 400. For under-reinforced beams, they give a higher value of the steel lever arm and a closer approximation to the true steel stresses.

It may be noted that the same method can be applied to T beams if it is determined from tests what the limiting effective proportions should be. The writer has not as yet made any examination of test data but there are probably enough on record to indicate how widely the uniform compressive stress in the concrete can be assumed to be distributed.

BEAMS REINFORCED FOR COMPRESSION

If there is steel in the compression side of the beam as in the case of a symmetrically reinforced arch rib, the compression steel will be stressed up to its elastic limit if the beam fails in compression. If it fails in the tensile steel, the presence of the compressive steel will have comparatively little effect on its ultimate strength. In the latter

TABLE 2—COMPARISON OF CALCULATED AND ACTUAL ULTIMATE STRENGTH OF BEAMS TESTED BY HUMPHREY AND LOSSE

Beam Numbers	Concrete			Steel				Max. Moment Actual (in. lbs.)	$\frac{c}{d}$	Max. Moment Calculated From Eq (5) (in. lbs.)	Maximum Moment Actual Divided by Calculated
	Aggregate	Age weeks	Cylinder Strength (lbs. per sq. in.)	A_s (sq. in.)	d (in.)	p	Yield Point (lbs. per sq. in.)				
162–163–164	Granite	4	3241	.393	10	.49	42,490	177,000	.9621	160,500	1.103
186–187–188		4	2807	.785	10	.98	43,043	321,000	.9115	307,500	1.043
237–238–239		4	3158	1.571	9¼	1.96	41,857	492,000	.8350	507,000	.970
171–172–173		52	5589	.393	10	.49	42,573	193,600	.978	163,500	1.183
195–196–197		52	5086	.785	10	.98	42,320	340,000	.952	319,000	1.066
234–235–236		52	4957	1.571	9¼	1.96	41,877	543,000	.895	545,000	.996
255–256–257	Limestone	4	2087	.393	10	.49	41,653	164,000	.9425	154,000	1.064
273–274–275		4	2600	.785	10	.98	40,797	291,000	.9095	289,000	1.006
321–322–323		4	2725	1.571	9¼	1.96	38,037	471,000	.815	451,000	1.044
246–247–248		52	4069	.393	10	.49	41,003	183,000	.9708	157,000	1.165
282–283–284		52	4360	.785	10	.98	41,413	317,000	.9451	307,000	1.033
330–331–332		52	4230	1.571	9¼	1.96	37,673	532,000	.890	487,000	1.092
333–334–335	Gravel	4	3441	.393	10	.49	43,447	169,000	.9635	164,000	1.030
357–358–359		4	3440	.785	10	.98	42,600	321,000	.9285	310,000	1.035
405–406–407		4	3376	1.571	9¼	1.96	38,633	504,000	.857	482,000	1.045
342–343–344		52	5186	.393	10	.49	44,140	173,000	.9754	169,000	1.022
366–367–368		52	5242	.785	10	.98	40,107	356,000	.9559	300,000	1.186
414–415–416		52	5467	1.571	9¼	1.96	38,643	533,000	.9120	512,000	1.040
417–418–419	Cinders	4	1629	.393	10	.49	37,357	163,200	.9337	138,000	1.182
441–442–443		4	1555	.785	10	.98	41,170	281,000	.8475	273,000	1.030
489–490–491		4	1643	1.571	9¼	1.96	37,887	403,000	.7315	376,000*	1.071
426–427–428		52	2944	.393	10	.49	38,343	172,600	.9624	145,000	1.188
450–451–452		52	2619	.785	10	.98	41,103	306,000	.9090	293,000	1.044
498–499–500		52	2763	1.571	9¼	1.96	38,047	488,000	.827	458,000	1.064
										Average	1.071 ± .05

*Determined by Concrete Strength, Eq. (6)

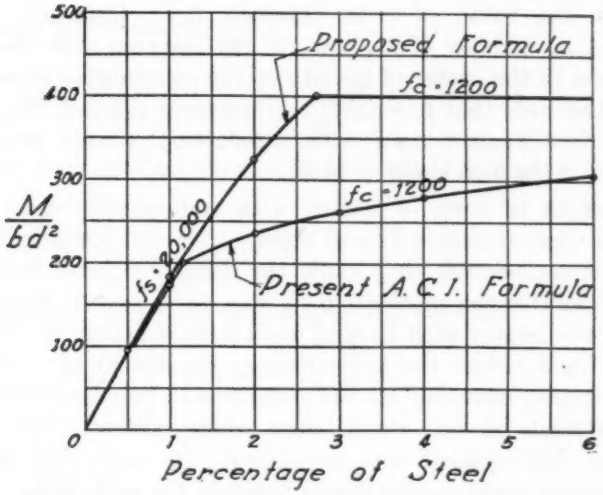


FIG. 4—COMPARISON OF RESISTING MOMENT OF RECTANGULAR BEAMS AS GIVEN BY PROPOSED FORMULA AND PRESENT A. C. I. FORMULA

case, the compressive steel can still be figured at its elastic limit if it comes within the compression zone and the lever arm of the tensile steel can be computed on that basis.

With compressive steel, the ultimate strength in compression is computed by adding the moment of the steel compressive stress to that of the concrete stress computed before. This condition is shown in Fig. 5.

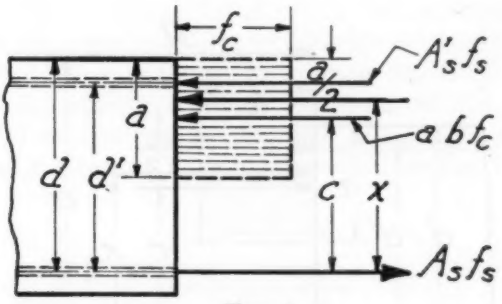


FIG. 5

The ultimate compressive moment will be

$$M_c = 0.333\,bd^2f_c' + d'A'_sf_s \quad \text{Eq. (9)}$$

The ultimate tensile moment will be

$$M_t = A_sf_s\,\frac{cabf_c + d'A'_sf_s}{abf_c + A'_sf_s} = x\,A_sf_s \quad \text{Eq. (10)}$$

in which the value of c is obtained from Equation (4) using $(M - d'A'_s f_s)$ in place of M, and x is the distance from the center of tensile steel to the center of gravity of the compressive stresses in the beam. Also note that $f_c = 0.85f'_c$. Equations (9) and (10) have also been checked against tests with satisfactory results provided the failure is not through shear or bond.

The results of tests of beams with compressive steel made by Bach and Graf at Stuttgart and reported by von Emperger [7, page 225] are of interest because they show the danger of bond failure. The full yield point stress of 35000 p.s.i. was developed in 20 mm. round bars but the beams with bars of steel with a 60,000 lb. yield point failed in bond before the full strength was developed. Glanville[12] advises against counting on the compression reinforcement but his tests were made on very small beams which evidently failed in shear before the full moment was developed. He says, "In the simple beams, serious cracking developed towards the end of the tests, and it is possible that the higher shear stresses in the continuous beams with compression reinforcement may have been the reason for the low moment carried over the central support."

FLEXURE AND DIRECT LOAD

The case of bending and direct stress can be treated in the same manner as bending alone. The strength of the compressive side will be the same as before and the steel tension will be reduced by the amount of the direct compression, P, as shown in Fig. 6.

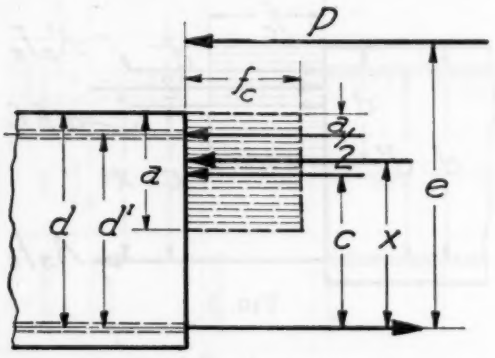


FIG. 6

The ultimate compressive moment is given by Eq. (9)

$$M_c = Pe = 0.333bd^2f'_c + d'A'_s f_s$$

The ultimate tensile moment is

$$M_t = Pe = A_s f_s \frac{x}{1 - \frac{x}{e}} \quad \text{Eq. (11)}$$

When there is no steel on the compressive side Equation (11) becomes

$$M_t = Pe = A_s f_s \frac{c}{1 - \frac{c}{e}} \quad \text{Eq. (12)}$$

and the value of c can be obtained from the formula

$$c = \frac{1}{2}\left[d + e - \sqrt{(d+e)^2 - 4de + 2Ye}\right] \quad \text{Eq. (13)}$$

in which

$$Y = \frac{A_s f_s}{bf_c} = \frac{A_s f_s}{0.85\, bf'_c}.$$

When there is steel on the compression side but the load is not sufficient to develop the full compressive strength of both the steel and concrete, the required tensile steel area can best be computed by considering the effect of the compression steel and concrete separately. First compute the area of tensile steel required to develop the moment ($d'A'_s f_s$) of the compressive stress with the formula:

$$A_s = A'_s \left(1 - \frac{d'}{e}\right) \quad \text{Eq. (14)}$$

Then deduct the moment ($d'A'_s f_s$) from the external moment, Pe, and compute the additional area of tensile steel required to balance the compression stress in the concrete using Eq. (12).

When the eccentricity is small compared with d, the ultimate strength of the member can be computed as twice the strength of the weaker side of the section from the formula (see Fig. 7):

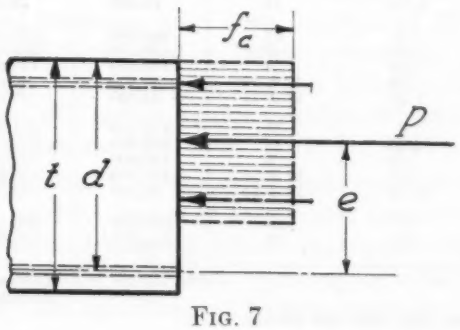


FIG. 7

$$P = 2A'_s f_s + 1.7b\,(d - e)\,f'_c \quad \text{Eq. (15)}$$

which becomes for a concentric load on a symmetrical member:

$$P = 2A'_s f_s + 0.85bf'_c \quad \text{Eq. (16)}$$

The value of e for which the Equation (9) and (15) will give equal values of P is given by Equation (17) and for smaller eccentricities, Equation (15) should be used.

$$e = \frac{1}{2}\left[d + Y' + \sqrt{(d + Y')^2 - 0.786d^2 - 2Y'd'}\right] \quad \text{Eq. (17)}$$

in which

$$Y' = \frac{A'_s f_s}{bf_c} = \frac{A'_s f_s}{0.85bf'_c}$$

The formulas for ultimate strength of members under flexure and direct load have been verified by comparison with the results of tests on 46 columns made by Bach and Graf [7][8]. The ratio of actual ultimate load to calculated ultimate strength is given in Table 3 which shows an average error of three per cent for the series. The columns were about 8 ft. 3 in. long and 16 in. square and were loaded with eccentricities varying from zero to about 20 in. The average 30 cm. cube strength of the concrete was 225 kg. per sq. cm. and

TABLE 3—COMPARISON OF CALCULATED AND ACTUAL ULTIMATE LOADS FOR COLUMNS TESTED UNDER ECCENTRIC LOADS BY BACH AND GRAF

(See JOURNAL Amer. Concrete Inst. *Proceedings* Vol. 26, p. 661. April, 1930.)

Column	Eccentricity	Calculated Ultimate Load		Actual Ultimate Load—Kg.	Actual Load Divided by Calculated Load
		Formula No.	Load—Kg.		
Type I	0	16	271,000	280,333	1.035
	.50	12	94,000	93,000	.990
	.75	12	57,000	60,333	1.057
	1.25	12	29,000	29,967	1.016
Type II	0	16	331,600	338,333	1.021
	.25	15	196,100	202,500	1.032
	.50	9	128,000	124,000	.968
	.75	12 & 14	69,800	69,600	.998
	1.25	12 & 14	32,800	32,350	.986
Type III	0	16	379,100	404,700	1.067
	.25	15	243,800	225,000	.923*
	.50	9	150,000	157,500	1.050
	.75	12 & 14	106,500	105,000	.987
	1.25	12 & 14	56,000	53,500	.955
Plain Columns	0	16	270,400	276,167	1.020
	.25	15	135,200	136,000	1.005
				Average	1.007
				Average Error	.031

*Average of two tests only, 0.882 and 0.965.

the cylinder strength was taken as $\frac{225}{1.13}$ or 199 kg. per sq. cm. The yield point of the steel averaged 3773 kg. per sq. cm. for Types I and II and 3572 for Type III.

The maximum variation in the strength of the cubes was about $\pm 9\frac{1}{2}$ per cent. The errors shown by the ultimate strength formulas are well within the limits of the variability of the materials and it would appear that the use of more elaborate formulas is not warranted.

For small eccentricities, Equation 15 agrees with test results much better than the usual formula based on straight line stress variation. Such agreement is more important than theoretical verification but it can be supported on the basis of actual stress variation. It is further supported by the results of tests on two plain concrete prisms reported by Slater and Lyse [2, page 859]. The 8 x 8 x 12 in. prisms were loaded with $e = \frac{t}{6}$ and the ultimate load averaged 155,520 lbs. The cylinder strength of the concrete was 4060 p.s.i. Eq. (15) gives

$$P = 1.7 \times 8 \times \frac{8}{3} \times 4060 = 147{,}000 \text{ lbs.}$$

or about 94.6 per cent of the actual load.

The foregoing equations predict the untimate strength of a member but cannot be used to compute the actual stresses below the ultimate. Being based on 85 per cent of the cylinder strength, they are consistent with the results of tests of columns under concentric load. It is hoped that they will provide a satisfactory method which is now lacking for the design of members of elastic frames and arches after the external moments have been determined by the theory of elasticity. They indicate that a higher percentage of tensile steel may be used than is now general practice without overstressing the concrete and they will permit lighter members where bond and shear can be properly taken care of.

It is further proposed that the dead load moments and thrusts be multiplied by a suitable factor of safety and added to the live load moments and thrusts multiplied by an appropriate factor, probably larger than that used for dead load, equating the total to the ultimate strength given by the equations. The factors of safety could also be

introduced by using reduced values of the stresses if it proves desirable. It may be that the ultimate strength formulas alone will be sufficient for ordinary building design without the necessity for making any calculation of the tensile stresses under working loads. A thorough study should be made to determine what factors of safety should be used for different types of structures.

REFERENCES

(1)"Final Report of the Special Committee on Concrete and Reinforced Concrete Arches," *Transactions*, Am. Soc. of Civil Engineers, 1935, p. 1427.

(2)"Compressive Strength of Concrete in Flexure," Slater and Lyse, JOURNAL Am. Concrete Inst., June 1930, *Proceedings*, Vol. 26, p. 831.

(3)"Reinforced Concrete Column Investigation, Tentative Final Report of Committee 105," F. E. Richart, Chairman, JOURNAL Amer. Concrete Inst., Feb. 1933, *Proceedings*, Vol. 29, p. 275.

(4)"Considerations sur le Clacul et la Sccurite des Pieces Flechies," by R. Copée, Publications of the International Association for Bridge and Structural Engineering, Vol. 3, p. 19.

(5)"Strength of Concrete Beams," by Richard L. Humphrey and Louis H. Losse, Technologic Papers of the Bureau of Standards.

(6)"Flexural Resistance of Shallow Concrete Beams," by C. B. McCullough, *Engineering New-Record*, Vol. 115, p. 406.

(7)"Handbuch fur Eisenbetonbau," von Emperger, Vol. 1, 3rd edition, p. 240.

(8)Discussion by George H. Dell, JOURNAL Amer. Concrete Inst., Apr. 1930, *Proceedings*, Vol. 26, p. 661.

(9)"Der Beiwert n," by Fritz v. Emperger, *Beton und Eisen*, Oct. 1936, p. 324.

(10)"Bruchzustand und Sicherheit im Eisenbetonbalken," by Rudolf Saliger, *Beton und Eisen*, Oct. 1936, p. 317.

(11)"Compressive Strength of Concrete in Flexure," by Slater and Zipprodt, *Proceedings*, Amer. Concrete Inst., 1920, p. 120.

(12)"The Redistribution of Moments in Reinforced Concrete Beams and Frames," by W. H. Glanville and F. G. Thomas, JOURNAL, Institution of Civil Engineers, June 1936, p. 291.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by June 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

(Supplement)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . SEPTEMBER-OCTOBER 1937

Discussion of a paper by Charles S. Whitney:

Design of Reinforced Concrete Members Under Flexure or Combined Flexure and Direct Compression*

CLYDE T. MORRIS†

The writer has been much interested in studying Mr. Whitney's paper. In addition to the references given at the end of his paper, a series of three articles on the same subject by Dr. Hajnal Konyi, appeared in the January, February and March issues of *Concrete and Constructional Engineering* (London) in which Dr. Konyi covers much the same ground as Mr. Whitney.

When applying the method to design it would seem to be better practice to use the nominal strength of the concrete in calculating the ultimate bending moments and dimensions given in equations (1) to (5) rather than 85 per cent of it as is done in Eq. (6). Then this 85 per cent as well as other uncertainties may be included in the factor of safety. In Europe the standard test specimens are cubes which have a crushing strength about $1\frac{1}{3}$ times that of the cylinders used in the United States, so that the .85 factor would have to be changed again when referring to European tests.

In Figs. 6 and 7, P is the resultant external force which is held in equilibrium by the internal stresses, therefore the arrow on the force P should be reversed. This is somewhat confusing when the sum of the horizontal components is taken for finding the stress in the tension steel.

When using the equations for design, it should be noted that Pe as given by the author is not the usual external moment used in column design. The moment usually calculated is referred to the axis of the column, and this must be increased by $\frac{1}{2}\ d'P$ in order to get the Pe used by Mr. Whitney.

*JOURNAL, Amer. Concrete Inst., Mar.-Apr. 1937, p. 483.
†Prof. of Civil Engineering, Ohio State University, Columbus.

A direct solution for the ultimate strength of a beam subjected to combined bending and direct compression may be obtained in a manner similar to that used in the ordinary theory of flexure.

Regardless of the shape of the stress strain curve, numerous tests have proved that plane sections before bending remain plane after bending, even for loads approaching the ultimate. Let Fig. 8 represent the conditions shown in the author's Fig. 6 with the change that e represents the distance from the resultant external force to the axis of the member.

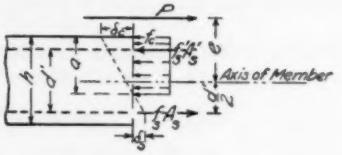


FIG. 8

The diagonal dotted line represents the plane section after bending and

δ_c = deformation of concrete just before failure

δ_s = simultaneous deformation of the tension steel.

If the eccentricity is small or the percentage of tension steel high, the value of f_s on the tension side may remain below the elastic limit. In that case $\delta_s = \frac{f_s}{E_s}$ and by moments about the tension steel,

$$P\left(e + \frac{d'}{2}\right) = f_c ab \frac{h + d' - a}{2} + f_s' A_s' d' \quad \ldots\ldots (18)$$

Also

$$P = f_c ab + f_s' A_s' - f_s A_s \quad \ldots\ldots (19)$$

and from similar triangles,

$$\frac{a}{\delta_c} = \frac{h + d' - 2a}{2\delta_s} \quad \text{or } f_s = \frac{(h + d' - 2a)\, E_s \delta_c}{2a} \quad \ldots\ldots (20)$$

By combining these three equations and eliminating P

$$f_c ab \frac{2e + a - h}{2e + d'} + f_s' A_s' \times \frac{2e - d'}{2e + d'} - E_s A_s \delta_c \times \frac{(h + d' - 2a)}{2a} = 0 \quad \ldots (21)$$

For any specific case it is possible to solve this equation for a and obtain a value for P from equation (19), but the process is long as equation (21) is a cubic.

In reinforced concrete arches the reinforcement is usually symmetrical and for this case equation (21) may be simplified somewhat

$$A_s = A'_s = pbh \qquad \frac{a}{h} = k$$

to correspond to the usual meaning of k in elastic theory.

A safe value of δ_c may be taken as about twice the yield point deformation of the steel. Then

$$E_s\delta_c = 2f_s' \text{ and}$$

$$k^3 + \left(\frac{2e}{h} - 1\right)k^2 + \frac{f_s'p}{f_c}\left(\frac{6e + d'}{h}\right)k = \frac{f_s'p}{f_c}\left(\frac{2e + d'}{h}\right)\left(\frac{h + d'}{h}\right) \quad (22)$$

Curves may be plotted for various percentages of reinforcement and several ratios of $\frac{d'}{h}$ and $\frac{f'_s}{f_c}$ showing the variation of k with $\frac{e}{h}$. Note that p in equation (22) represents the percentage of steel in each side of the member. Figure 9 shows such curves for $f'_s/f_c = 12$ and $\frac{d'}{h}0.8$.

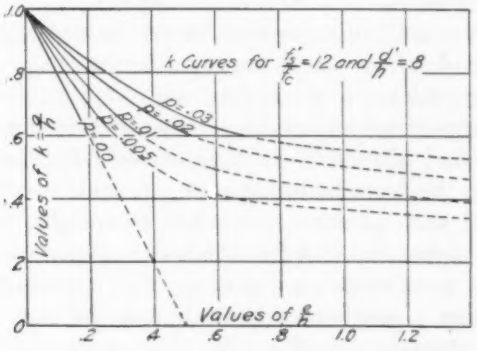


FIG. 9

If the eccentricity is large or the amount of tension steel insufficient, failure will occur when the tension steel reaches the yield point. If we assume that the concrete deformation δ_c is twice the yield point deformation of the steel when the yield point of the tension steel is reached, by proportion,

$$k = \frac{a}{h} = \frac{1}{3}\left(1 + \frac{d'}{h}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

If k from equation (22) is less than this amount failure will occur by the tension steel reaching the yield point. In this case the steel, both on the tension and compression sides is at the yield point when the concrete reaches its ultimate strength. By moments in Fig. 8 about the resultant P,

$$f_c ab\left(e-\frac{h}{2}+\frac{a}{2}\right)+f_s'A_s'\left(e-\frac{d'}{2}\right)-f_sA_s\left(e+\frac{d'}{2}\right)=0 \quad \ldots\ldots (24)$$

This is a quadratic equation with a as the only unknown and agrees with Mr. Whitney's Eq. (13). When the reinforcing is symmetrical $A_sf_s = A_s'f_s'$

$$a=\frac{h}{2}-e+\sqrt{\left(e-\frac{h}{2}\right)^2+\frac{2f_s'A_s'd'}{f_cb}} \quad \ldots\ldots (25)$$

With this value of a the ultimate value of P may be obtained from Eq. (19).

Factor of Safety

Forty years ago steel structures were designed with a, so called, "factor of safety." This was usually based on the ultimate tensile strength of the material and had little real significance, as the ultimate carrying capacity of a structure designed with a nominal factor of safety of 4 was seldom more than twice the design load. In those days the factor of safety was in reality a factor of ignorance and was used largely by salesmen in dealing with public officials who had little or no engineering knowledge. Due to this fact the term fell into disrepute and allowable working stresses for design were substituted. Now, the term factor of safety is seldom used. In dealing with a material such as steel which has a straight line relationship between stress and strain and for which the properties do not change with time and conditions of exposure, the permissible unit stress method of design is probably preferable to the use of a factor of safety on the ultimate strength. But for concrete structures there seems to be considerable merit in the suggestion that designs be made for ultimate loads with suitable factors of safety. The first problem to be solved is the selection of appropriate factors for various conditions of loading.

Factor of Safety for Dead Load

The proper factor of safety for dead loads depends upon the possible variation between the assumed design strength of the concrete and the strength of the actual material in the structure, and also upon the degree of accuracy with which the dead loads can be predicted.

Tests have shown, as stated by Mr. Whitney, that the ultimate strength of concrete in laboratory beams and columns is only about 85 per cent of the cylinder strength of the same concrete, and it is fair to assume that if this ratio holds in laboratory tests, the ratio between the concrete in the field and laboratory cylinders is much

lower. On the other hand, concrete increases in strength as it ages and this increase in strength may be utilized for loads applied at ages greater than 28 days which is the usual age for testing control cylinders. However it is not safe to assume that the application of the dead load will be delayed beyond the 28 day period. It is not unusual for control cylinders made under the same specifications in the same structure to vary 10 per cent either way from their mean. Therefore the available ultimate strength of the concrete in the structure may not exceed (85 per cent — 10 per cent) $\frac{3}{4}$ = 56 per cent of the assumed cylinder strength. The factor $\frac{3}{4}$ is used as a possible ratio between assumed laboratory concrete and field concrete in the structure at 28 days.

Dead Load Estimate

Dead Load estimates are usually made from design drawings and subsequent changes in details, deflection of forms, and inaccuracies in construction may increase the dead load beyond the original estimate. Especially is this true of girders and columns in multi-storied buildings.

In 1924 Prof. A. H. Fuller of Iowa State College published the results of an investigation carried on during the erection of the Equitable Building in Des Moines, Iowa.* The stresses were measured in four steel columns during the erection of the building by means of strain gages. Professor Fuller states "Mr. Stegenga's computations for loads were checked against the design data, and it is believed errors above 5 per cent have been eliminated." The observed and computed dead loads on the four columns were as follows, in p.s.i.

Column No.	Observed	Computed	Ratio $\frac{\text{Obs.}}{\text{Comp.}}$
5	8500	7840	1.08
6	9300	6570	1.42
20	7300	5220	1.40
28	7400	4420	1.67

The variation in the overrun of the dead load among these four columns seems excessive for estimates from finished design drawings. In 1925-26 a similar investigation was undertaken by A. Ward Ross, Jr., under the supervision of the writer, during the erection of the 48 story building for the American Insurance Union in Columbus, Ohio.** In this case stresses were measured on 8 of the main columns under the tower and the ratios of observed to computed dead load were as follows: 1.20, 1.31, 1.23, 1.29, 1.27, 1.26, 1.22, and

*"Measurements of Streases in 4 Steel Columns of the Equitable Building, Des Moines, Iowa" by A. H. Fuller, Bulletin No. 72, Engr. Exp. Sta. Iowa State College, Ames, Iowa.

**"Dead Load Stresses in the Columns of a Tall Building" by Clyde T. Morris, Bulletin No. 40 Engr. Exp. Sta., Ohio State University, Columbus, Ohio.

1.30 or an average of 26 per cent overrun. The dead loads were estimated from the finished detail drawings and the loads present were checked by a survey once a week during the progress of construction. The errors in estimates from original rough design sketches might exceed those obtained in this investigation. It would therefore seem that actual dead loads might overrun the original estimates by 30 per cent to 40 per cent. This would give a minimum allowable factor of safety for dead load of $\frac{1.3 \text{ or } 1.4}{.56} = 2.3$ to 2.5.

Factor of Safety for Live Loads

The discussion of concrete strengths given above under the heading of Factor of Safety for Dead Loads applies in all respects to live loads with the possible exception of the age of the concrete at which it is reasonable to expect the application of a maximum live load. It is probably safe to assume that the maximum design live load will not be applied to the structure until the concrete is at least six months old, and it is also reasonable to assume that the strength of the concrete will have increased some 20 per cent during the period from 28 days to six months. This would change the effective concrete strength to (85 per cent — 10 per cent + 20 per cent) $\frac{3}{4}$ = 71 per cent of the assumed 28 day cylinder strength.

Estimate of Live Load

In estimating the live load to be provided for, possible future changes in the nature of the occupancy, provisions for side sway due to moving loads, and impact must be taken into consideration.

In industrial buildings, improvements in methods of manufacture may require the installation of new, or the rearrangement of old machinery, increasing the live load; or in any building a change in the nature of the occupancy may entirely alter the live load requirements.

In tier buildings originally designed for office or residential purposes, changes in conditions surrounding the location may change, entirely, the uses to which the building will be put. Such a building, ten years after its erection may be used for light manufacturing or storage purposes.

In railway and highway bridge specifications provision for future overload is always required, usually at an increased allowable unit stress. If the design is to be based on the ultimate strength, such provisions for overload must be included in the factor of safety. The American Association of State Highway Officials in their specifications for Highway Structures provide for a future increase in live load

of 100 per cent. Highway and railway bridge specifications also provide for impact by adding to the static live load stress a percentage depending upon the nature of the loading. This practice does not need to be altered in using the present proposed method of designing for ultimate loads with a factor of safety. However, in buildings there is usually no definite knowledge of the magnitude of the impact factor and this must be included in the factor of safety.

From the above discussion of live loads it would appear that the magnitude of the factor of safety is somewhat dependent upon the nature of the structure under consideration, but that a factor of 2 for the assumed live load would be a minimum for any structure. This would give a factor of safety for live loads of $\frac{2.00}{0.71} = 2.82$. Summarizing the entire discussion, it would appear that a factor of safety for both dead and live loads of 3 is not too conservative.

As stated by Mr. Whitney, the use of this method in design "indicated that a higher percentage of tensile steel may be used than is now general practice." It might also indicate the economy of using reinforcing steel with a higher yield point.

The differences in design procedure and in the results obtained is illustrated in the following calculations for a simple rectangular beam.

Assume a beam of 24-ft. span carrying a dead load of 1200 lb. per lin. ft. and uniform live load of 800 lb. per lin. ft. Assume the concrete to have a cylinder strength of 3000 p.s.i. and the steel to have a yield point strength of 36,000 p.s.i. $f_c = 1200$ p.s.i. $f_s = 18{,}000$ p.s.i.

Conventional Design

Dead Load Mom. $= \frac{wL^2}{8} =$ 86.4 ft. kips

Live Load Mom. $=$ " $=$ 57.6 " "

Total M $=$ 144.0 " "

Assume a beam width (b) of 12 in. $n = 10$

Approximately $M = \frac{f_c b d^2}{6} = 144 \times 12$ from which

$d^2 = 720 \qquad d = 26.83$

Use a beam 12" x 29" with effective depth $d = 27$"

Approximately $A_s = \frac{M}{\frac{7}{8} d \times f_s} = \frac{144 \times 12}{\frac{7}{8} \times 27 \times 18.0} = 4.06$ sq. in.

Use 2 bars $1\frac{7}{16}$" sq. $= 4.13$ sq. in. $\qquad p = \frac{4.13}{27 \times 12} = 0.0128$

$k = 0.394$ $\qquad j = 0.869$

$M = \frac{1}{2} f_c k j b d^2$ $\qquad f_c = 1150$ p.s.i.

$$f_s = \frac{M}{jdA_s} = 17{,}830 \text{ p.s.i.}$$

Design for Ultimate Strength

Using a factor of safety of 3 for both dead and live loads

Ultimate Moment $= 3 \times 144 = 432$ ft. kips

Assume a beam width (b) of 12 in.

For approximate depth use Eq. (6)

$$d^2 = \frac{3M}{f_c' b} = 432 \qquad d = 20.8 \text{ in.}$$

Use a beam 12″ x 23″ Effective depth $d = 21$ in.

From Eq. (3) $a = 8.63$ in.

From Eq. (1) $A_s = 8.63$ sq. in.

Use 4 bars $1\frac{1}{2}$″ square. $A_s = 9.00$ sq. in.

If a steel with a yield point of 48,000 p.s.i. were used,

$A_s = 6.47$ sq. in. and 4 bars $1\frac{3}{16}$″ square will be sufficient.

The results of this comparison for a simple beam design illustrates the rather startling possibilities of saving in dead load.

Combined Bending and Direct Compression

The crown section of a barrel arch is 15 in. deep and reinforced with 1 per cent of steel $1\frac{1}{2}$ in. from each face. The moment per ft. width at the crown is 36,000 ft. lb. and the thrust 30,000 lb. By the usual elastic theory the maximum calculated fiber stress in the concrete is 1060 p.s.i.

Assuming concrete with an ultimate strength of 3000 p.s.i. and a steel with an elastic limit of 36,000 p.s.i.

$$e = \frac{36{,}000}{30{,}000} = 1.2 \text{ ft.} = 14.4'' \qquad \frac{e}{h} = \frac{14.4}{15} = .96$$

From Fig. 9, $k = 0.44$ which is less than the value obtained from Eq. (23) and therefore failure will occur first in the tension steel.

From Eq. (25) $a = 2.6$ in. and from Eq. (19) $P = 93{,}600$ lbs.

$$\text{Factor of safety} = \frac{93{,}600}{30{,}000} = 3.12.$$

If the percentage of steel were doubled, $a = 4.7$ in. and $P = 169{,}200$ lbs., giving a factor of safety of 5.64. This shows the possibilities of economy in concrete by increasing the percentage of steel.

L. J. MENSCH*

Mr. Whitney should be commended for having made an effort to induce American engineers to abandon the time-worn and incorrect straight line theory for the design of reinforced concrete members under flexure and under combined loading. He is the first American engineer who had the courage to do so since the writer presented a paper on this subject to the Institute in 1915.† The author evidently never saw this paper or other discussions in many volumes of the *Transactions* of the Am. Soc. C. E. but he noticed some oversea papers by authors who were very late in taking up this subject.

The present paper, although laudable, is not based on sufficient evidence, nor does it contain advice how to put the new ideas to practical use. The writer will endeavor to supply part of these deficiencies, especially, as very few copies of his original paper are now in existence.

To consider the upper part of a reinforced concrete beam at the instant of approaching failure to be in uniform ultimate compression was the idea which the pioneers of reinforced concrete had in mind when they tried to analyze their tests to destruction. Ransome, Coignet, Hyatt, Hennebique did so. Dr. Emperger taught it in the Vienna Technische Hochschule 40 years ago. The astounding success of Hennebique induced some writers to invent a so-called scientific analysis—the straight line theory—which was enthusiastically adopted all over the world and considered Bible truth.

The assumption of uniorm ultimate compression is rather primitive and leads to considerable inaccuracies in plain sections and in reinforced concrete meimers such as round columns and in members under combined loading therefore, the writer first adopted a parabolic stress distribution and later one according a cubic parabola, which by no means complicates the resulting formulas.

The writer welcomes the assumption of 0.85 f'_c as the ultimate compressive strength of concrete in bending; Mr. Whitney shows considerable spirit in putting himself on record in opposition to nearly all so-called concrete experts.

The assumption that the tensile stress in the reinforcing steel at the time of approaching failure is the yield point (no definition given) is not in agreement with the most careful tests made by experienced investigators. The author seems to have overlooked the excellent tests by D. B. Steiman‡ or by T. D. Mylrea** in the same issue of

*Consulting Engineer and Contractor, Chicago.

†"New-Old Theory of Reinforced Concrete Beams in Bending," by L. J. Mensch, presented 1915 Convention. JOURNAL, of Amer. Concrete Inst., December 1914, p. 28, *Proceedings* Vol. 10.

‡"Isteg Steel for Concrete Reinforcement," by D. B. Steinman, JOURNAL Amer. Concrete Inst., Nov.-Dec., 1935; *Proceedings* Vol. 32, p. 183.

**"Concrete Slabs Reinforced with Welded Wire Fabric," by T. D. Mylrea, JOURNAL Amer. Concrete Inst., Nov.-Dec. 1935; *Proceedings* Vol. 32, p. 219.

this JOURNAL and his later tests, privately published by the Rail Steel Bar Association, or the tests by Inge Lyse and George H. Wernisch*, or by H. J. Gilkey and G. C. Ernst (*Proc. Highway Research Board*, Dec. 1935), or by the German Concrete Committee (Bulletin 38, 1917), or those by Dr. Emperger and many others, or he would have known that in many of these tests the ultimate stress of the reinforcing steel in tension was found greater than the commercial yield point and nearly equal to the ultimate tensile strength, when this stress was obtained by dividing the ultimate bending moment by The Total Depth of the Beam and not by the computed j.d only.

From all the tests which the writer could find he came to the conclusion that the stress in the reinforcing steel in a beam at the ultimate load in a testing machine is the stress when the unit strain reaches the value of 0.005 and a good and conservative guess for these stresses is:

43000 p.s.i. for mild steel.
52000 " " intermediate grade steel
72000 " " ordinary rail steel
100000 " " hard rail steel in small sizes.

Fig. 10 shows our assumptions, and considering that the area of a cubic parabola = ¾ times the area of the enclosing rectangle and that its center of gravity is 0.4 kd. from the top, we can write

Compression = Tension, or $p\,b\,d\,f_s = \frac{3}{4}\,b\,kd \times 0.85\,f_c'$

whence, $k = 1.57\,p\,f_s/f_c'$ ..... (26)

The ultimate moment equals Force times Leverage

$$M = b\,d\,p\,f_s\,jd = bd^2\,p\,f_s\,(1 - 0.4k) = b\,d^2\,p\,f_s\,(1 - 0.6275\,p\,f_s/f_c') \quad (27)$$

By observing the cracks of several hundred highly reinforced beams at the time of failure the writer concludes that the depth of the compression zone can never become greater than 0.8 d., how-so-ever high the percentage of tensile reinforcement may be. The steel will attain the ultimate useful limit stress and the greatest compression will exist in the concrete when, according to Eq. (26)

$$1.57\,p\,f_s\,/\,f_c' = 0.8$$

from which we obtain the balanced or critical percentage of reinforcing as

$$p = 0.51\,f_c'/f_s \quad (28)$$

and from Eq. (27) we obtain then the maximum possible moment which a highly reinforced beam can withstand

*"A Study of Reinforcement in Concrete Slabs," by Inge Lyse and George Wernisch, JOURNAL, Amer. Concrete Inst., Sept.-Oct. 1936, Vol. 33 p. 1.

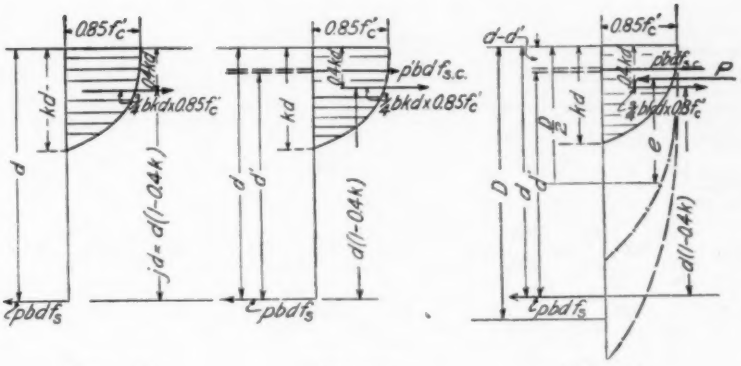


FIG. 10 FIG. 11 FIG. 12

$$M = b\,d^2 \times 0.51\,f_c' \,(1 - 0.6275 \times 0.51) = 0.346\,b\,d^2 f_c' \ldots\ldots\ldots\ldots\ldots (29)$$

While this formula is based on many hundred tests made by the writer, it is quite remarkable that the author from only a few tests obtained the value of $M = 0.333\ b\ d^2 f_c'$ which varies only by 4 per cent from Eq. 29). Both formulas are derived from tests and not from theory, hence they must agree when the basic tests are properly conducted.

The advantage of Eq. (27) cannot readily be seen, therefore, Table 4 has been prepared for four grades of steel and for six grades of concrete. It shows at a glance that the compressive strength of the concrete plays only a very small role for low percentages of tensile reinforcement, but it also shows that for high percentages of reinforcing, especially of high strength steel, the ultimate capacity of a reinforced concrete beam is greatly dependant on the concrete compressive strength and is directly proportionate to it at the balanced reinforcement.

For ¼ per cent of reinforcement, for example, and for intermediate grade steel, M/bd^2 increases only 8 per cent when 5000 lb. concrete instead of 1000 lb. concrete is used, which means that concrete only a few days old will make as strong a beam as at any time in the future and that we can remove forms in a few days. Slabs reinforced with ½ per cent of 72000 lb. steel have their ultimate M/bd^2 increased only 23 per cent when the concrete strength increases from 1000 lb. to 5000 lb., while beams reinforced with 2 per cent of 72000 lb. steel have the M/bd^2 increased 71 per cent when the concrete strength increases from 2000 lb. to 5000 lb.

C. B. McCullough (*Eng. News-Rec.*, Sept. 19, 1935) shows how confusing the present practice is; the writer never used the straight

TABLE 4
(Table continued opposite page)

f_s =	43000								52000								72000			
f'_c =	1000		2000		3000		5000		1000		2000		3000		5000		1000		2000	
f_s/f'_c	43		21.5		14.33		8.6		52		26		17.33		10.4		72		36	
% of Tension Reinforcement	k / j	M / bd^2	k / j	M / bd^2	k / j	M / bd^2	k / j	M / bd^2	k / j	M / bd^2	k / j	M / bd^2	k / j	M / bd^2	k / j	M / bd^2	k / j	M / bd^2	k / j	M / bd^2
¼%	.169 .933	100	.084 .967	104	.056 .977	105	.034 .987	106	.204 .917	119	.102 .959	125	.068 .974	127	.041 .984	128	.282 .887	160	.141 .944	170
½%	.337 .865	186	.169 .932	200	.112 .955	205	.068 .973	209	.408 .833	216	.204 .918	238	.136 .945	245	.082 .967	251	.565 .774	278	.282 .887	319
1%	.675 .73	313	.338 .865	371	.225 .91	386	.135 .946	406	.8 .68	346	.408 .837	435	.272 .891	463	.164 .934	485	.80 .68	346	.565 .774	556
1½%	.80 .68	346	.505 .798	514	.338 .865	558	.202 .919	592	.8 .68	346	.611 .756	590	.407 .837	652	.245 .902	703	.80 .68	346	.80 .68	692
2%	.80 .68	346	.675 .739	635	.45 .82	705	.27 .892	767	.8 .68	346	.8 .68	692	.544 .782	813	.327 .869	904	.80 .68	346	.80 .68	692
3%	.80 .68	346	.8 .68	692	.675 .739	942	.405 .838	1081	.8 .68	346	.8 .68	692	.8 .68	1038	.49 .804	1250		346		692
4%	.80 .68	346	.8 .68	692	.80 .68	1038	.54 .784	1345		346		692	.68	1038	.653 .739	1538		346		692
5%	.80 .68	346	.8 .68	692		1038	.675 .73	1570		346		692		1038	.80 .68	1730		346		692
6%	.80 .68	346	.8 .68	692		1038	.80 .68	1730		346		692		1038	.80 .68	1730		346		692

Tension Reinforc

line theory and is not able to make a comparison between its results and those of Table 4; possibly the author or some other discusser will undertake the task.

The writer had difficulties in following the author's discussion of reinforced concrete beams with top reinforcement. There is, besides, a considerable error in the assumption that the compressive stress in the top reinforcement at the approach of failure equals the tensile stress of the bottom reinforcement. The writer has shown (A. C. I. JOURNAL, Dec. 1914) that the ultimate stress in mild steel used as top reinforcement is only about 32000 p.s.i., and in high carbon steel about 50000 p.s.i. The research paper No. 873 of the National Bureau of Standards, March, 1936, shows that the ultimate strength of structural steel when embedded in concrete is only 36000 p.s.i., but the ultimate strength of the embedding concrete figured then only ⅔ of the cylinder strength, therefore, it would be better to assume

72000				100000												72000		72000		72000	
3000		5000		1000		2000		3000		4000		5000		10000		3000					
24		14.4		100		50		33.33		25		20		10		24					
k	$\frac{M}{bd^2}$	k	$\frac{M}{bd^2}$	k	$\frac{M}{bd^2}$	k	$\frac{M}{bd^2}$	k	$\frac{M}{bd^2}$	k	$\frac{M}{bd^2}$	k	$\frac{M}{bd^2}$	k	$\frac{M}{bd^2}$	Compression Reinforcement					
j		j		j		j		j		j		j		j							
.094	174	.057	176	.393	210	.196	230	.131	236	.098	240	.079	242	.039	246	p′ = 1%		p′ = 2%		p′ = 3%	
.962		.977		.843		.922		.948		.961		.969		.984							
.188	333	.113	343	.785	343	.393	421	.261	447	.197	460	.157	468	.079	484						
.925		.955		.686		.843		.895		.921		.936		.969							
.376	612	.226	655	.80	346	.785	686	.521	792	.392	843	.314	874	.157	933	k	$\frac{M}{bd^2}$	k	$\frac{M}{bd^2}$	k	$\frac{M}{bd^2}$
.85		.91		.68		.686		.792		.843		.874		.933							
.565	835	.339	934		346	.80	692	.781	1030	.588	1147	.470	1215	.235	1359	.303	960		960		
.774		.864				.68		.688		.765		.812		.906							
.753	1005	.452	1179		346		692	.80	1038	.785	1372	.628	1498	.314	1748	.492	1195	.230	1300		1300
.699		.819						.68		.686		.749		.874							
.80	1038	.678	1574		346		692		1038	.80	1384	.80	1730	.471	2433	.80	1488	.608	1780	.346	1920
.68		.729								.68		.68		.811							
	1038	.80	1730		346		692		1038		1384		1730	.628	2996	.80	1488	.80	1938	.722	2330
		.68												.749							
	1038	.80	1730		346		692		1038		1384		1730	.785	3430						
		.68												.686							
	1038	.80	1730		346		692		1038		1384		1730	.80	3460						
		.68												.68							

ement only

Tension & Compression Reinforcement

the ultimate strength of intermediate grade steel when used as top reinforcement also at 32000 p.s.i.

By inspection of Fig. 11:

$$p\,b\,d\,f_s = p'\,b\,d\,f_{s.c.} + \tfrac{3}{4}b\,kd \times 0.85\,f_c' \qquad (30)$$

or

$$k = 1.57\,\frac{p\,f_s - p'\,f_{s.c.}}{f_c'} \qquad (31)$$

and

$$M = p'\,b\,d\,f_{s.c.}\,d' + 0.6375\,k\,b\,d^2\,f_c'\,(1 - 0.4\,k) \qquad (32)$$

Again, k, from Eq. (31) cannot become greater than 0.8 or Eq. (32) would not be valid as f_s would be less than the ultimate useful limit stress, which we have assumed as 72000 p.s.i. for ordinary rail steel bars, for example. Eq. (31) also shows that the assumption of the author of $f_s = f_{s.c}$ is improbable; for symmetrical reinforcing it would

lead to a $k = 0$ which is impossible. Where kd is less than twice the distance $d - d'$ the compression steel is not strained to the ultimate value, the stress is reduced in proportion to the ordinates of the concrete stress parabola and we have to develop new formulas as the following reasonings show:

We have to first assume a value for k and to ascribe to the stress of the compression reinforcement the value

$$f'_{s.c.} = f_{s.c.} \left[1 - \left(\frac{d - d'}{kd}\right)^3\right] \quad \ldots\ldots (33)$$

and

$$p\,b\,d\,f_s = \tfrac{3}{4}\,b\,d\,k\,0.85\,f_c' + p'\,b\,d\,f'_{s.c.} \quad \ldots\ldots (34)$$

and

$$M = p'\,b\,d\,f'_{s.c.}\,d' + 0.6375\,b\,kd\,f_c'\,(1 - 0.4\,k)d \quad \ldots\ldots (35)$$

The best way to make use of equations (30) to (35) is to construct a table which an engineer can readily apply; a very short one is given in Table 4 in the last 6 columns.

As an example for the application of Eq. (30) to (32) assume

$p = 2$ per cent, $p' = 1$ per cent $d' = 0.9\,d$, $f_s = 100{,}000$,

$f_{s.c.} = 50{,}000$, $f_c' = 3000$

From Eq. (31) $k = 1.57\,\dfrac{2000 - 500}{3000} = 0.785$

From Eq. (32) $M = bd^2 \times 500 \times 0.9 + 0.6375 \times 0.785 \times 3000\ (1 - 0.785 \times 0.4)]$

$M = 1460\,b\,d^2$ against $1038\,b\,d^2$ as given in Table 4. when no top reinforcing is used. For $p' = 2$ per cent we similarly obtain $M = 1692\,b\,d^2$.

As an example for the use of Eq. (33) to (35) assume p' = 2 per cent $f_s = 72000$, $f_{s.c.} = 50{,}000$, $f_c' = 3000$ and $k\,d = 1\tfrac{1}{2}(d - d') = 0.15\,d$.

From Eq. (33) and (34) we shall determine the proper p at which the tensile reinforcing will just be stressed to its ultimate value for the assumed kd.

From Eq. (33) $f'_{s.c.} = \left[1 - \left(\dfrac{0.1\,d}{0.15\,d}\right)^3\right] f_{s.c.} = 0.704\,f_{s.c.}$

From Eq. (34) $72000 \times p = 0.02 \times 0.704 \times 50000 + 0.6375 \times 0.15 \times 3000$

whence, $p = 1.38$ per cent.

and from Eq. (35)

$M = b\,d^2\,[0.6375 \times 0.15 \times 3000\ (1 - 0.06) + 0.02 \times 0.704 \times 50000 \times 0.9]$

$M = 910\,b\,d^2$

By assuming a somewhat larger kd we would obtain a larger p. Mr. Whitney assumed $f'_{s.c.}$ to be always equal to f_s, regardless of the value of k, therefore, his formulas would lead to considerable variations from facts, especially in round members.

The author's treatment of reinforced concrete members under combined loading of direct compression and bending is affected by the same error previously mentioned in regard to the ultimate compressive

stress of the steel and, besides, he offers some formulas without derivation and adopts an excentricity, o, with respect to the tensile reinforcement instead of the customary e with respect to the center of gravity of the section; his formulas are applicable to a rather restricted field.

We have to distinguish 3 principal cases:

1—The eccentricity, e, of the excentric force P, as shown in Fig. 12 is so small that the entire section is in compression, also the bottom reinforcement, and the apex of the cubic parabola is outside the section, as shown by the dashed line. This generally happens when the excentricity is smaller than 0.15 d for $p' = 3$ per cent and less than 0.1 d when p' is about 1 per cent.

The writer has shown elsewhere (*Trans. Am. Soc. C. E.*, Vol. 86, 1923, page 1195) that a nearly exact formula for P is in this case

$$P = \frac{b\,D \times 0.85\,f_c' + p\,b\,d\,f_{s.c.} + p'\,b\,d\,f_{s.c.}}{1 + 3.14\,e/d} \quad \ldots\ldots (36)$$

2—The apex of the cubic parabola is inside the section but less than 0.2 d from the tensile reinforcement, as shown by the dot and dash line in Fig. 12. The maximum stress of the tensile reinforcement at the ultimate load is then less than the value f_s assumed in the girder theory. This case happens for an excentricity of 0.1 d to 0.3 d, depending on the shape of the section and the kind of reinforcing. Formula (36) gives good working values which are correct within a few percent.

3—The excentricity of P is so great that the apex of the cubic parabola is farther from the tension reinforcement than 0.2 d; f_s will now attain its maximum possible value, corresponding to a strain of 0.005 inch per inch.

The writer has treated the case of the excentrically loaded column very extensively in a paper which was to have been presented at the 1918 meeting of this Institute; it was never presented on account of the absence of the writer and was not published. A similar paper for steel columns may be found in the *Transactions of the Am. Soc. C. E.*, Vol. 83, 1919-20, page 1667.

For this third case we can write down the following equations:

$$P + p\,b\,d\,f_s = p'\,b\,d\,f_{s.c.} + \tfrac{3}{4}\,b\,k\,d \times 0.85\,f'c \quad \ldots\ldots (37)$$

$$\text{whence,} \quad k = 1.57\,\frac{P/bd + p\,f_s - p'\,f_{s.c}}{f'c} \quad \ldots\ldots (38)$$

and taking moments about the center of gravity of the section which without great error, as a rule, we may take as mid-point of the total depth, D,

$$M = P\,e = \tfrac{3}{4}\,b\,kd\,0.85\,f'_c\,(\tfrac{1}{2}D - 0.4\,kd) + p'\,b\,d\,f_{s.c}\left[\tfrac{1}{2}D - (d - d')\right] + p\,b\,d\,f_s\,(d - \tfrac{1}{2}D) \quad \ldots\ldots (39)$$

Eq. (37 to (39) are only valid when kd is greater than twice $(d - d')$, otherwise, the reduced value $f'_{s.c.}$ must be introduced as shown in case of the concrete beams with top reinforcement.

The practical way to make use of formulas (37) to (39) is to construct a table which comprises all cases likely to occur in practice. Such a table may be found for steel columns in *Trans.*, Am. Soc. C. E., Vol. 83, above mentioned, and the unpublished paper for this Institute contained a table for plain and reinforced concrete columns, round and square; a diagram based on these tables may be found in *Trans.* Am. Soc. C. E. ,Vol. 86, 1923, page 1195.

As an example, let $f_s = 52000$ p.s.i., $f_{s.c.} = 32000$ p.s.i.; $p = p' = 0.02$; $f'_s = 3000$ p.s.i.; $d' = 0.9\,d$, $D = 1.111\,d$.

From formula (38) $$k = 1.57\frac{P/bd + 400}{3000} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40)$$

and from formula (39)

$$P\,e = b\,d^2\left[\tfrac{3}{4} \times 0.85 \times 1.57\,(P/bd + 400)\left(0.555 - 0.6275\,\frac{P/bd + 400}{3000}\right)\right]$$
$$+ 0.02\,b\,d^2\left[32000\,(0.555 - 0.1) + 52000\,(1 - 0.555)\right]$$
$$P\,e = b\,d^2\left[(P/bd + 400)\left(0.555 - 0.6275\,\frac{P/bd + 400}{3000}\right) + 754\right] \quad \ldots\ldots\ldots\ldots (41)$$

The easiest way to construct a table for this case is to set, for example,

	$P = 500\,bd$,	$1000\,bd$,	$1200\,bd$.
from Eq. (40)	$k = 0.523$,	0.723,	0.80
from Eq. (41)	$e = 1.98\,d$,	$1.11\,d$,	$.915\,d$

Intermediate values may be found by interpolation.

A concentrically loaded column would be able to support an ultimate load of $3000 \times 0.85 \times b \times 1.111\,d + 0.04\,b\,d \times 32000 = 4370\,b\,d$, hence, $P = 500$, 1000 and $1200\,b\,d$, correspond to 11.5, 23 and 27.5 per cent of the concentric load bearing capacity.

A value of $P = 1600\,b\,d$ would correspond to a kd larger than d for which case formula (36) would apply.

The author claims that his formulas (9) to (17) agree exceedingly and surprisingly well with Bach & Graf's tests. The writer's formulas not only agree with these tests but with all other excentric column tests which have been published.

GEORGE ROBERT WERNISCH*

Any suggestion for the elimination of that highly unknown quantity in reinforced concrete design—the modular ratio—should be given careful consideration. The author has shown that his method of equivalent stress block, when applied to actual test data, is in error not in excess of six per cent in the calculation of the concrete resisting moment. Anyone familiar with the design and placing of concrete will agree that this accuracy is at least as good, if not better, than that obtained in the concrete used in ordinary construction.

It should be noted that Mr. Whitney is not assuming rectangular stress distribution, but is merely substituting an equivalent rectangular stress block for the supposed parabolic stress distribution. The method, therefore, is rational and sensible and should not be discounted merely because it is a radical departure from our present straight-line theory.

Although the strain may vary lineally in a reinforced concrete beam, experimental work has indicated that the stress distribution is not linear; it may be approximately so under working loads, but

*Formerly Concrete Reinforcing Steel Institute Research Fellow, Lehigh University, Bethlehem, Pa.

plastic flow should cause some variation even at low loads. Professor Westergaard, in an able analysis of stresses at a crack*, applying Airy's stress functions and Herz' theory of pressures between two spherical bodies in contact, also indicated that the stress distribution was far from linear. This analysis convinced the writer that, considering the continually changing slope of the stress-strain diagram of concrete and its plastic characteristics, it would not be altogether impossible to obtain a condition of stress distribution which would very nearly approximate rectangular dimensions.

The important point in Mr. Whitney's contribution is his attempt to obtain a rational design in which the ultimate concrete and steel resisting moments are equal. His method enables one to obtain a truly "balanced" design in which failure of steel and concrete occurs simultaneously.

The writer was particularly interested in the limiting value of a which is the distance from the top of the beam to the neutral axis. Mr. Whitney takes this distance as 0.537d, and states that it is independent of p, d, and f_c. He is intimating, in other words, that our present method of calculation of k is entirely out of line with experimental results. The writer has made numerous concrete modulii tests and the value of n calculated therefrom was considerable higher than that obtained from the present formula. Of course, plastic flow, a somewhat indeterminate quantity, will also increase the modular ratio.

There are others who feel that our present method of calculation of the neutral axis is highly questionable. Professor Lyse, in a recent contribution to *Beton u. Eisen*† using some of the data obtained in the Slater-Lyse tests, has shown (Fig. 13) that a fifth degree parabola

*Journal American Concrete Institute, Sept.-Oct. 1934, *Proceedings* V. 31, p. 90.
†Apr. 5, 1937

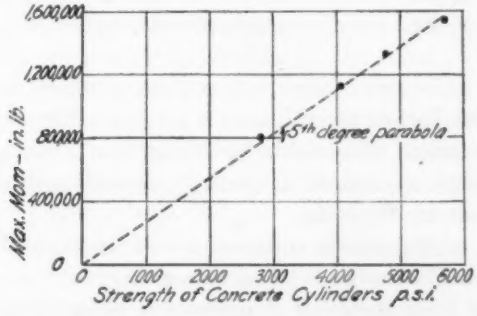


FIG. 13—EFFECT OF STRENGTH OF CONCRETE ON STRENGTH OF REINFORCED CONCRETE BEAMS

stress distribution approximates actual ultimate load stress conditions fairly satisfactorily *when the neutral axis is assumed to be at the center of the beam.* Professor Lyse states: "It is noted that except for very lean concrete, the maximum moment of failure increased very uniformly with the increase of the strength of the concrete, indicating that the position of the neutral axis remained stationary."

The author has shown that his equation (6) $M/bd^2 = f'c/3$ is correct within several per cent of actual conditions; its simplicity of application is obvious.

In Fig. 14 the writer indicates the difference between the Whitney and Lyse methods. The Whitney method would require slightly less concrete and slightly more steel than the method outlined by Professor Lyse. It should be noted that either method would probably require somewhat more steel and considerably less concrete than the present method of design. Mr. Whitney probably believes that the increased cost of the steel will be more than offset by the savings in the cost of the concrete. So far as concrete is concerned, the Whitney method

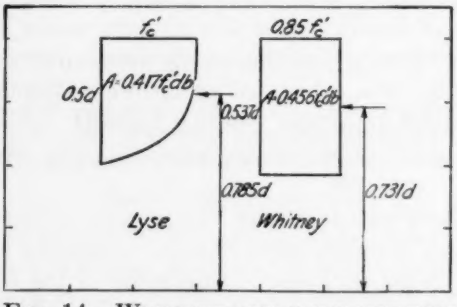


FIG. 14—WHITNEY AND LYSE METHODS

leads to a very economical use thereof. The science of concrete making has progressed to such an extent that a smaller factor of safety than that which has been employed heretofore can be used without any danger.

The author also has a very convenient method of determining c (the moment arm) when the moment is known. The writer has applied this method to some tests made by Slater and Lyse and the Bureau of Standards and compared it with observed and calculated steel stresses as shown in Table 5.

The Bureau of Standards tests seem to corroborate Mr. Whitney's statement that his method gives lower steel stresses for under-reinforced beams than the present method of calculation. The tests of Slater and Lyse indicate that for heavily reinforced beams there is a fair agreement between the straight line and Whitney methods for

TABLE 5

(Slater and Lyse Tests)

Group	Moment	A_s	$f'c$	d	j	c/d	Observed Stress	Whitney Stress	Straight Line Stress
2	741,000	2.37	2790	10.3	0.823	0.814	39,400	37,300	36,900
3	1,011,800	3.14	4070	10.3	0.827	0.815	39,200	38,300	37,800
4	1,281,600	3.92	4800	10.1	0.820	0.789	42,000	40,600	39,400
5	1,461,600	4.71	5700	10.2	0.820	0.812	38,600	37,500	37,100

(Bureau of Standards Typical Tests: $f_s = 41{,}000$ p.s.i. $f_c = 4000$ $d = 10$ in. $b = 8$ in.)

Moment	Steel	j	c	Straight Line Stress	Whitney Stress
176,000	0.4	0.917	0.974	48,100	45,200
246,000	0.6	0.907	0.960	45,200	42,700
320,000	0.8	0.893	0.948	44,800	42,200
380,000	0.9	0.888	0.938	47,700	45,000
420,000	1.04	0.880	0.929	45,700	43,500
468,000	1.24	0.873	0.921	43,200	41,000
540,000	1.39	0.863	0.907	45,000	42,800

partial load conditions, even though the Whitney formula is intended primarily for ultimate load conditions; this may or may not be significant. The author undoubtedly contemplates the use of larger percentages of steel and probably there will be little difference between the stresses calculated by either method. It should be noted that the larger percentages do not necessarily mean larger steel areas; a larger percentage is obtained due to the decreased section which the author contemplates.

The author should be complimented for not involving his method by incorporating the complicated theory of Saliger relative to steel stresses. Professor Saliger gives us a plausible reason why observed steel stresses are usually lower than calculated steel stresses in concrete structures (because the yield point of the steel is usually reached only at cracks, the concrete between the cracks aiding in resisting tension; thus a local yielding at the crack, when taken over a ten-inch gage length would have only a slight effect on the average stress. If measurements could be taken over one or two-inch gage lengths the observed stresses would be found to be considerably higher than those found over a ten-inch gage) but his suggested method of computation is too involved for practical application. Inasmuch as Mr. Whitney's method of calculation would ignore the tensile aid given by the concrete, it is on the safe side and approximates the actual steel stress fairly well except in the case of thin slabs, and there should be no objection thereto. It may be more economical, however, to design thin slabs under-reinforced.

Although he does not specifically mention it, Mr. Whitney's proposal would indicate he is in favor of designing on the basis of ultimate strength rather than working stresses, and dividing by the desired factor of safety in order to determine the working load. There should be no objection to this method.

Mr. Whitney's use of the value $0.85f'_c$ in the beam formula may be open to discussion. However, this value is not assumed to be the actual maximum stress in the beam, but the average over the depth of the equivalent stress block, "a". The use of this value has two advantages. It corresponds with the column strength obtained from test data (thus making possible a consistent treatment of all cases of bending and direct stress) and it appears to give a suitable value of "c" which checks tests for flexure alone, and bending and direct stress.

The proposed method outlined by Mr. Whitney should tend to encourage the use of stronger concretes, especially in those cases where the designer is confronted with the question of initial cracking, which is dependent upon the modulus of rupture of the concrete. The use of higher concrete strengths will decrease the costs considerably, especially in those structures where the dead loads is large.

Incidentally, if more engineers would realize that great savings in reinforced concrete construction can be realized only by using stronger concretes there would be less agitation for higher steel working stresses, the use of which probably will do more harm than good when all factors are considered.

BRUCE JOHNSTON*

The writer does not wish to question or discuss the merits of the proposed new assumptions for designing concrete in flexure. Attention should be called, however, to the stress-strain curve in Fig. 1, p. 485, which exhibits a prolonged drop in stress near the ultimate with respect to increasing strain. Insofar as the writer knows, typical stress-strain curves for concrete do not usually exhibit this drop. The falling off of stress near the top of the beam in flexure, also illustrated in Fig. 1, is dependent on the unusual type of stress-strain curve used by the author.

The preceding statements do not necessarily mean that the proposed method does not give a much closer approximation to the truth than the more usual triangular assumption of stress distribution. Before adopting such a design method, however, the writer

*Instructor in Civil Engineering, Columbia University, New York City.

believes that more thorough study should be given to the actual distribution of stress in a beam under flexure, as based on typical stress-strain curves for concrete in compression. Methods for determining the distribution of stress in flexure for any arbitrary stress-strain curve are given in Nadai's "Plasticity," Chapter 23, page 164.

The author of the paper has brought to the attention of American Engineers a worthwhile subject for further consideration.

INGE LYSE*

The paper by Mr. Whitney brings before the Institute members a problem which has recently been thoroughly discussed in foreign magazines, particularly those in Germany. *Beton und Eisen* has during the last few years, contained a number of articles on this problem by some of Europe's most prominent engineers. Such names as Emperger, Saliger, Gebauer, Bittner, Stüssi and Steuermann are connected with these new methods of design. An excellent contribution on the so-called Saliger method is given in English by Dr. K. Hajnal-Konyi in the January, February, and March 1937 issues of *Concrete and Constructional Engineering.* A brief presentation of Professor Saliger's method of analysis is also given in Dr. Hajnal-Konyi's discussion in the January-February 1937 issue of this JOURNAL†. The interest in a more correct method of analyzing the strength of reinforced concrete beams and slabs is therefore very wide and it is hoped that Mr. Whitney's paper will create extensive discussion.

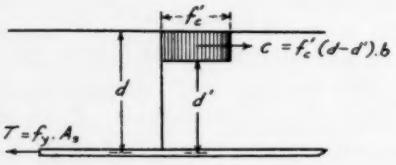
The type of deformation for plain concrete, shown in Fig. 1 of the paper, does not seem probable to the writer. It does not appear reasonable that the compressive stress in a beam reaches a maximum at a certain strain and then decreases as the strain increases. The stress distribution shown in Fig. 1 is therefore questionable. The use of 85 per cent of the cylinder strength as a strength basis in the beam analysis is also questioned.

With our present lack of information regarding the actual stress distribution in reinforced concrete beams near the ultimate strength, it seems that approximate computation methods which give reasonable strength values are justified. The well known rectangular distribution method for the analysis of ordinary reinforced concrete beams in which the yield-point stress of the reinforcement is always reached before failure occurs, offers, as shown by Mr. Whitney, a simple method of design. As the stress in the steel reaches the yield-point value deformation takes place without marked increase in the

*Research Professor of Engineering Materials, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.

†Discussion by Hajnal-Konyi of a paper by Lyse and Wernisch (Sept.-Oct. 1936, p. 1, this volume) "A Study of Reinforcement in Concrete Slabs."

steel stresses. This causes the position of the neutral axis to move closer to the compressive surface as the cracks in the tension side become greater. Finally the compressive area becomes so small that the concrete crushes. Thus the actual failure in all reinforced concrete beams is due to the crushing of concrete, provided a ductile reinforcing steel is used. Fig. 15 shows the method of computation for this condition.



$$T = C = f_y . A_s = f'_c (d-d')b$$

$$M' = T\left(d - \tfrac{1}{2}(d-d')\right) = \tfrac{1}{2} T(d+d') = \tfrac{1}{2} f_y . A_s (d+d')$$

$$d-d' = \frac{f_y . A_s}{f'_c . b} \quad ; \quad d' = d - \frac{f_y . A_s}{f'_c . b}$$

FIG. 15—STRESS CONDITION AT FAILURE OF UNDER-REINFORCED BEAMS

A comparison between the maximum moments computed by this simple method and the observed moments at failure is shown in Fig. 16, both for a series of beam tests at Columbia University reported by Hajnal-Konyi in his English paper, and for a series of slab tests at Lehigh University. The agreement between computed and observed values is seen to be satisfactory.

When excessive reinforcement is used so that the yield point of the steel has not been reached at the time of failure, which is a condition analyzed by Mr. Whitney, the above method does not apply directly. In this case we have to determine the location of the neutral axis of the beam. For overreinforced beams* the neutral axis is located close to the mid-height of the beam, that is, k = about 0.5. If a fifth degree parabola is assumed for the distribution of compressive stresses in the concrete we obtain the following j value:

$$j = 1 - \frac{r+1}{2(r+2)} \cdot k = 1 - \frac{5+1}{2(5+2)} \cdot \frac{1}{2} = \frac{11}{14}$$

where r is the degree of the parabola.
We have then:

$$M_{max} = \frac{5}{6} \cdot \frac{11}{28} f'_c \cdot bd^2 = \frac{55}{168} f'_c \cdot bd^2$$

or approximately:

*See "Der Beiwert n im Eisenbetonbau," by Inge Lyse in *Beton und Eisen*, Heft 7, April 5, 1937.

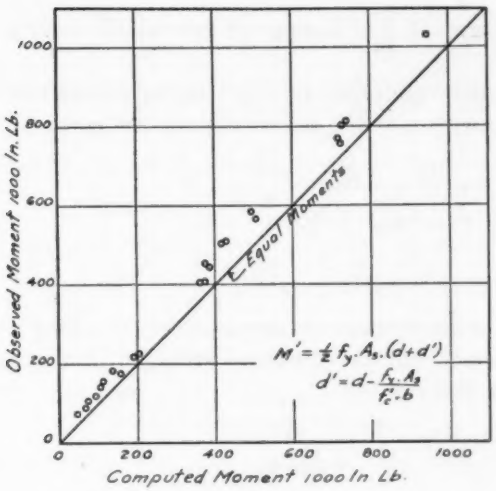


FIG. 16—COMPARISON OF COMPUTED AND OBSERVED MOMENTS AT FAILURE

$$M_{max} = \frac{1}{3} f'_c \cdot bd^2$$

which is the equation for rectangular distribution. The maximum stress in the steel becomes:

$$f_s = \frac{M_{max}}{j \cdot d \cdot A_s} = \frac{M_{max}}{\frac{11}{14} \cdot d \cdot A_s}$$

Table 6 gives the comparison between the steel stresses observed at a load close to the ultimate and the stresses computed for this load.

TABLE 6—STRESS IN REINFORCEMENT

Group No.	Concrete Strength	Maximum Load	Measurements at Last Observation			
			Load	Strain in Steel	Observed Stress	Computed Stress
	p.s.i.	lb.	lb.		p.s.i.	p.s.i.
1	1390	32,520	30,000	0.00116	33,700	38,400
2	2790	44,710	40,000	.00136	39,400	38,400
3	4070	63,300	55,000	.00135	39,200	39,900
4	4800	74,830	70,000	.00145	42,000	40,000
5	5740	87,410	80,000	.00133	38,600	38,400

Except for the beam having concrete of very low strength, the computed stresses agree very well with the observed stresses.

A moment arm of $\frac{11}{14}d$ thus gives reasonable values for the stress in the steel in overreinforced beams. Using this as the limiting value for the determination of the reinforcement we have:

$$\tfrac{1}{2}(d + d') = \frac{11d}{14} = 0.785d$$

$$d + d' = 1.57d$$

or

$$d' = 1.57d - d = 0.57d$$

The amount of reinforcement required for reaching the yield-point stress in the steel and the ultimate strength of the concrete simultaneously is then given by:

$$A_s = \frac{M'}{f_y \cdot \tfrac{1}{2}(d + d')} = \frac{M'}{0.785\, d \cdot f_y}$$

An amount of reinforcement greater than this would not utilize the full yield value of the steel and thus be uneconomical. Since the moment at failure for overreinforced beams is approximately $\frac{1}{3} f'_c . bd^2$, we have the following expression for the maximum area of reinforcement:

$$A_{s_{max}} = \frac{\frac{1}{3} f'_c \cdot bd^2}{0.785d \cdot f_y} = 0.42 \frac{f'_c}{f_y} \cdot bd$$

or

$$p_{max} = 0.42 \frac{f'_c}{f_y}$$

which is slightly less than the amount

$$\left(p_{max} = 0.456 \frac{f'_c}{f_s} \right)$$

recommended by Mr. Whitney for design purposes.

Fig. 17 shows the agreement between computed and observed maximum moments when the five-degree parabolic stress distribution is used. Again the agreement is very satisfactory for ordinary strength concrete. It should be kept in mind that the failure of these beams was due to crushing of the concrete before the yield point of the steel was reached.

The writer therefore wishes to endorse the use of the rectangular stress distribution for the computation of the ultimate strength of reinforced concrete beams and slabs until more correct methods have been established by experimental studies.

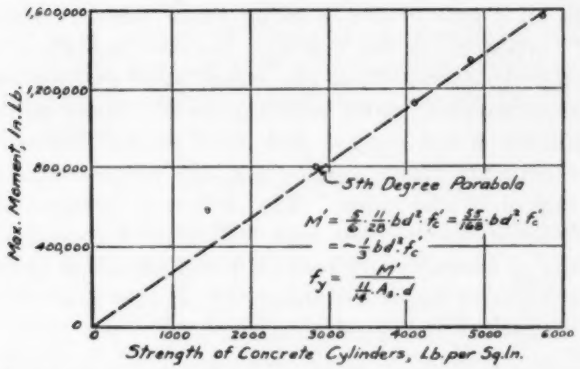


FIG. 17—EFFECT OF STRENGTH OF CONCRETE ON STRENGTH OF OVER-REINFORCED BEAMS

L. C. URQUHART*

Mr. Whitney's paper should lead to a large amount of much needed discussion concerning the fundamentals of reinforced concrete design. His treatment, while perfectly logical, is in the end so simple, especially in contrast with that of other recent articles† in the same direction, that it furnishes an excellent basis for the complete revision of method which he suggests.

The concrete in a beam designed by the proposed method when reviewed by the "straight line" method has an apparent stress of 60 per cent in excess of the working value (Fig. 18), and a stress of

*Professor of Structural Engineering, Cornell University, Ithaca, N. Y.
†References 9 and 10 of original paper.

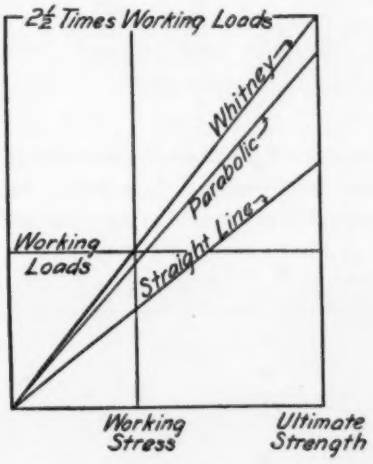


FIG. 18

10 per cent in excess of the working value when reviewed by the "parabolic" method. In the first case the steel is slightly more than 10 per cent understressed, and the second case slightly less than 10 per cent understressed under working loads. Since neither of the above assumptions are correct and since safety factors should be based upon ultimate stress distribution, the proposed method seems the more logical of the three. The difference between it and the "parabolic" being only ten per cent, which is, of course, well within any variation of assumed and realized live load seems again to favor its use on account of its greater simplicity, and its lack of use of, that so difficult to assume correctly value, n.

The question of the economy of the resultant members designed by the proposed method over those designed in accordance with present practice is bound to enter into a good deal of the discussion. While in light construction and shallow members, subject to flexure only, the economy is negligible, in heavy construction with deep members the saving in dead load weight is evident. In the former design loads are often never realized and the exact distribution of the stresses at any stage of loading is not important. In the latter the importance of the actual distribution of the stresses should be understood and considered by the designer. Here, also the saving in weight is considerable, the proposed method allowing a reduction of thirty per cent in the concrete section, although it does require an increase of seventy per cent in the steel area.

To the writer it seems more desirable to retain working stresses as a basis of design. To do this, of course, a thorough study of the proper safety factors must be made. It would seem that for some types of loading no difference need be made between the factors for dead and live load while for other types of loading different factors may be desirable.

Since so many investigations have shown that the ultimate strength in an actual member is somewhat less than the cylinder strength, the author is fully justified in using the factor of 0.85. Another method of arriving at the same result would be to change the definition of concrete strength and specify that f'_c is to be taken equal to 0.85 of the strength of cylinders at the age of 28 days. This results in Eq. (6) becoming $\frac{M}{bd^2} = 0.4\, f'_c$, with f'_c as above, or with a factor of $2\frac{1}{2}$, $\frac{M}{bd^2} = 0.16\, f_c$.

The development for beams with compression reinforcement and that for members subject to direct stress combined with flexure, follows so easily and logically that it is really simpler than the present standard practice, as well as exemplyfying stress distribution more closely to the actual.

AUTHOR'S CLOSURE

THE writer has made a thorough re-study of his recommendations in the light of the discussions submitted and does not suggest any change in the formulas. The discussors seem to agree that some such simplified method for the design of concrete members subjected to flexure is practical and desirable. They have added greatly to the value of the paper by clarifying various points and have given the writer this opportunity to present additional supporting data.

Professor Lyse and Mr. Johnston have questioned the shape of the stress-strain curve in Fig. 1 which shows a drop in stress near the extreme compression fiber. Saliger's study* indicated this drop but showed that it does not occur until the beam is very near failure. As stated before, the exact shape of the stress distribution curve for the beam is not important. Whatever the shape of the stress curve may be, there must be an equivalent rectangular stress block which can be used for design purposes. The strength of a fully reinforced concrete beam is independent of the shape of this stress block as Eq. (6) shows that it is a direct function of f'_c alone. The lever arm of the steel and the amount of steel needed for balanced reinforcement are however determined by the shape of the stress block and any assumed shape is suitable if it establishes these values satisfactorily. In order to check the suitability of the shape assumed by the writer, the results of tests of 337 beams have been plotted in Fig. 19, covering the full practical range of concrete and steel strengths and percentage of reinforcement. A description of the tests is given in the appendix. The reinforcement varies from less than 10 per cent to over 150 per cent of p_o, the theoretical amount needed for balanced reinforcement, and the tests therefore indicate the accuracy of both Eq. (6) and Eq. (7).

Correction

Equation (42) page 498-27 should be:

$$\frac{M}{b\,d^2 f'_c} = 0.456\,\frac{p}{p_o}\left(1 - 0.268\,\frac{p}{p_o}\right) \qquad (42)$$

Changes in the shape of the assumed stress block are suggested by Professor Morris, Professor Lyse, and Mr. Mensch. The writer wishes to indicate what the effect of these changes would be.

The use of $0.85\ f'_c$ as the value of f_c is questioned by Professor Morris, Professor Lyse and Mr. Wernisch and approved by Professor Urquhart and Mr. Mensch. If f_c is made equal to f'_c the strength of the beam as given by Eq. (6) is not effected but the length of the steel lever arm, c, is increased and the required amount of steel is decreased. For the condition of balanced reinforcement this changes c from $0.732d$ to $0.788d$ and the steel area from $0.456\frac{f'_c}{f_s}$ to $0.423\frac{f'_c}{f_s}$. The curve representing the assumption that $f_c = f'_c$ is shown in Fig. 19 by the dash line and lies above a considerable number of the test points. It is less conservative and does not fit the tests quite as well as the formula with $f_c = 0.85\ f'_c$. Since the strength of concrete varies with the shape of the structural unit tested, there is no reason for assuming that the strength of the concrete in the beam is the same as the strength in the cylinder. Also it should be remembered that f_c is not the maximum stress but represents the average stress in the assumed rectangular stress block. The writer prefers to use the value of $f_c = 0.85\ f'_c$ which is consistent with column tests.

Discussing the case of a beam which is not under-reinforced, Professor Lyse shows that the use of a fifth degree parabola as the stress distribution curve with $f_c = f'_c$ and the neutral axis at the center of the beam ($k = 0.5$) results in an equation approximately equal to Eq. (6). However, the resulting value of c is $0.785\ d$ and $p_o = 0.42\frac{f'_c}{f_s}$, which are almost identical with the values given by the rectangular stress block assumption with $f_c = f'_c$. The fifth degree parabola would therefore be subject to the same objections as the latter assumptions. The writer does not agree with Professor Lyse that it is necessary to determine the location of the neutral axis and that the same method does not directly apply to both over-reinforced and under-reinforced beams. In checking with tests the value of the steel lever arm, c, for the case of balanced reinforcement, only those tests are significant in which the value of p is fairly close to p_o. The two points representing the U. S. Geological Survey tests of 22 beams in Fig. 19 lie close to the writer's curve and indicate that 0.785 is too high for the value of $\frac{c}{d}$ for the case of balanced reinforcement. It should be kept in mind that dependable test data are rather meager and the

differences between these different suggested formulas are not great.

Mr. Mensch suggests the use of a third degree parabola for the stress distribution curve with the neutral axis at eight-tenths of the depth of the beam ($k = 0.8$) for balanced reinforcement. This results in the value of c equal to $0.68d$ instead of $0.732\ d$ and the steel requirement $p_o = 0.51\frac{f'_c}{f_s}$ compared with the writer's $p_o = 0.456\ \frac{f'_c}{f_s}$. The bending moment is nearly the same as that given by Eq. (6) but slightly higher. Mr. Mensch compensates for his low steel lever arm by using an arbitrarily higher value of steel yield point stress. The writer has not seen any steel stress-strain curves which show as high yield point stresses as those suggested by Mr. Mensch even when the unit strain is 0.005. Saliger and other authorities recommend that the yield strength of the steel be defined as the stress producing a unit strain of 0.004 and that value has been widely used in connection with tests with Isteg and other steel without definite yield points. If Mr. Mensch were to use more usual values of yield point strength his formulas would require an excessive amount of steel.

The writer is familiar with the tests mentioned by Mr. Mensch which developed much higher than yield point stresses in the reinforcement. Dr. Steinman's tests at Columbia University are plotted in Fig. 19 and show reasonably good agreement with the formulas. The other tests were made on slabs or very small beams. For several reasons the writer does not believe that the excess of observed strength of slabs over the theoretical strength should be recognized in design.

Prof. T. D. Mylrea suggests* that the excess strength exhibited by under-reinforced specimens loaded quickly to failure may not be a phenomenon to be depended on in design. In the case of a slab subjected to several repetitions of load, the first visible crack occurred at a lower computed steel stress than in the other slabs of the group. Holding the load which produced a calculated stress equal to the ultimate strength of steel tension coupons eventually ruptured the bars.

Marked excess strength is observed only in slabs which are greatly under-reinforced and it is probable that better structures will result if the use of more steel rather than less is encouraged.

The articles by Dr. K. Hajnal-Konyi in Concrete and Constructional Engineering mentioned by Professor Lyse and Professor Morris present a very interesting discussion of the defects of the standard method of design, an account of new methods recently proposed in

*"Concrete Slabs Reinforced with High Yield Point Steel Bars," Highway Research Board Proceedings, 1936, p. 100.

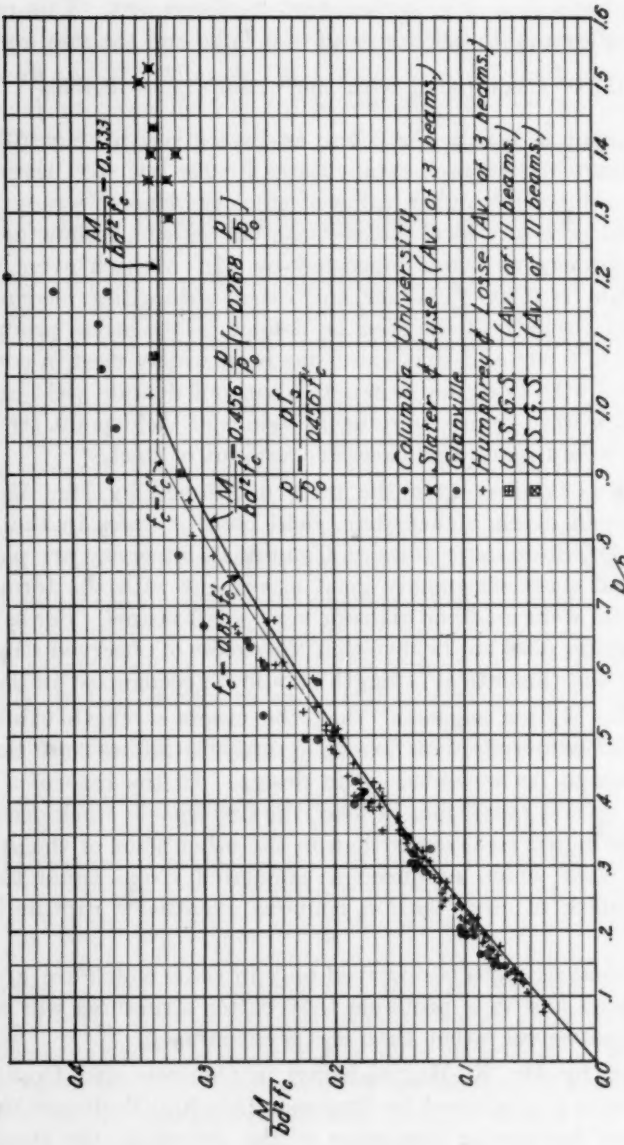


FIG. 19—THEORETICAL ULTIMATE STRENGTH OF CONCRETE BEAMS COMPARED WITH RESULTS OF 337 TESTS

Europe, and a description of Professor Saliger's method which he considers to be the best of the new methods. It appears to the writer that Professor Saliger has unnecessarily complicated his formulas through an attempt to take into consideration the excess strength shown by under-reinforced slabs.

Mr. Mensch apparently does not understand the writer's equations for beams with compression reinforcement which may need further explanation. It is not necessarily assumed that the stress in the top steel is the same as that in the bottom steel although it would be so for the case of balanced reinforcement. For under-reinforced beams the compression in the top steel may even be neglected if it is not needed to help develop the full strength of the tensile steel. Any conditions of stress may be assumed on the compressive side which will develop the tensile steel and at the same time give the greatest c distance, provided the maximum stresses are not exceeded. The figures quoted by Mr. Mensch indicate that the yield point strength as ordinarily defined may be developed in the compression steel.

Tests made with Isteg bars in the compression side of beams indicate that because of buckling due to the twisting of the bars, they are not fully effective in compression as they are in tension.

Mr. Mensch is correct in stating that the concrete strength has comparatively little effect on the ultimate flexural strength of beams with very light reinforcement but his conclusion that the forms could therefore be stripped at an early date is not necessarily justified. With the weaker green concrete, the useful strength of the beam would be a smaller proportion of the ultimate flexural strength because of lower shearing strength and a greater tendency for the beam to distort and crack.

Regarding the formulas for combined bending and direct stress, there appears to be no advantage in assuming a parabolic stress distribution curve instead of a rectangular stress block. Mr. Mensch deserves much credit for the work he has done in the field of concrete design and his preference for his own formulas is understandable but his closing remark indicates that he has not tested Equations (9) to (17).

Mr. Wernisch has mentioned the value of a as being the distance to the neutral axis. It is not used in that sense by the writer. As indicated in Fig. 1, the equivalent stress block does not extend to the neutral axis and no relation is established between them.

It should also be noted that Professor Morris has assumed a as the distance to the neutral axis in the derivation of his formulas which are therefore not directly comparable with the writer's formulas.

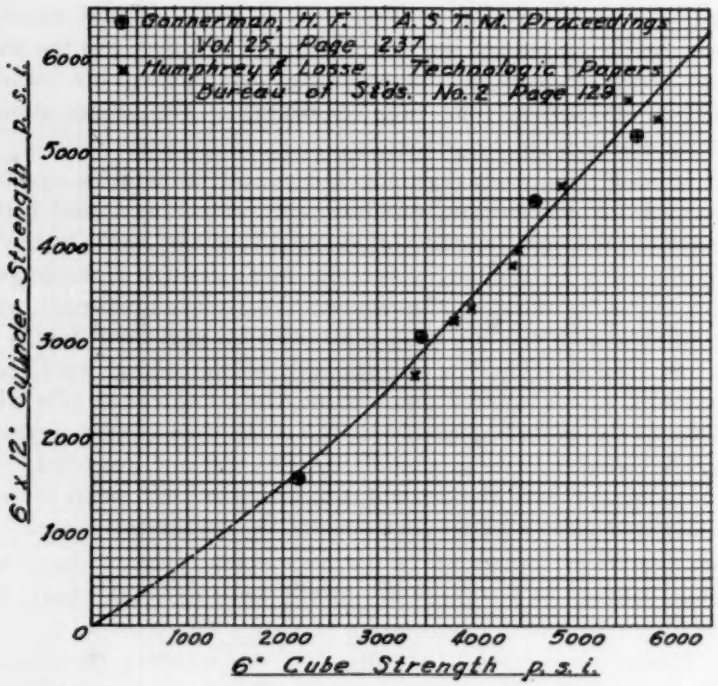


FIG. 20—RELATION BETWEEN STRENGTH OF 6 X 12-IN. CYLINDER AND 6-IN. CUBE

Professor Morris' formulas are based on further limiting assumptions as to strains in the concrete and steel which may not always be justified and the resulting formulas are somewhat more involved. The writer can see no objection to the use of e to represent the eccentricity of load measured from tensile steel in as much as it appears to simplify the design procedure.

Professor Morris suggests that the 0.85 factor expressing the relation between f_c and f'_c be omitted from the formulas and placed in the factor of safety. This cannot be done even if it were desirable because its effect on under-reinforced beams is variable depending on the relative amount of reinforcement. It applies only to concrete stresses and not to steel stresses, whereas the factor of safety applies to both.

In applying the writer's formulas to European practice, it is necessary to translate the strength of standard cubes into the strength of 6 by 12-in. cylinders. Fig. 20 shows this relation for 6-in. cubes as indicated by some of the best tests but there are probably other variables which make this translation rather unsatisfactory. It appears probable that the cylinder is a much more satisfactory index

of the strength of the concrete in a beam or column than is the cube. It may therefore be more difficult to obtain formulas which will be as satisfactory for European practice as long as the cube is used as a standard.

Professor Morris' discussion of factor of safety is valuable and interesting. Eliminating the $f_c = 0.85\ f'_c$ factor, his value of 3 for the factor of safety becomes about 2.5 which is about what the writer arrives at by an entirely different process of reasoning. If the loads are conservatively estimated and f'_c is taken as the minimum strength of field cylinders, it seems that the useful strength of concrete members might be conservatively calculated as forty percent of the ultimate strength making proper allowance for fatigue, repeated loadings, distortion, and inaccuracies in calculating bending moments. Special conditions may warrant using a higher or slightly lower factor of safety than 2.5.

Professor Urquhart expresses a preference for the use of working stresses and as many engineers may feel the same way, it should be pointed out that the formulas can be used directly for design by using working stresses instead of ultimate strengths. It is even possible by that method to use a somewhat larger factor of safety for the concrete than for the steel.

Professor Morris' examples of beam design are not very favorable to the new design method because he has assumed a steel with a yield point lower than that usually used for concrete reinforcement. The result is that his conventional beam actually has a factor of safety of 2.15 compared with a factor of safety of 3 for the smaller beam. Also he makes no reduction in moment due to the smaller dead load.

Fig. 21 is a comprehensive design chart for rectangular beams giving the values of $\frac{M}{bd^2}$ and percentages of steel for a full range of concrete and steel strengths. It indicates the great simplification in design procedure.

The writer has applied his formulas to Frame No. 1 tested by Richart and Olson* and calculated the ultimate strength to be 32,000. The average measured strength was 31,900 pounds.

The beam formulas can be applied to T beams but the included flange width should not be as great as 6 or 8 times its thickness as permitted when using the standard straight-line formula. As designed by the standard formula, a wide-flanged T-beam has a much lower

*"Rapid and Long-Time Tests on Reinforced Concrete Knee Frames," by F. E. Richart and T. A. Olson, JOURNAL, Amer. Concrete Inst., March-April, 1937; *Proceedings* Vol. 33, p. 459.

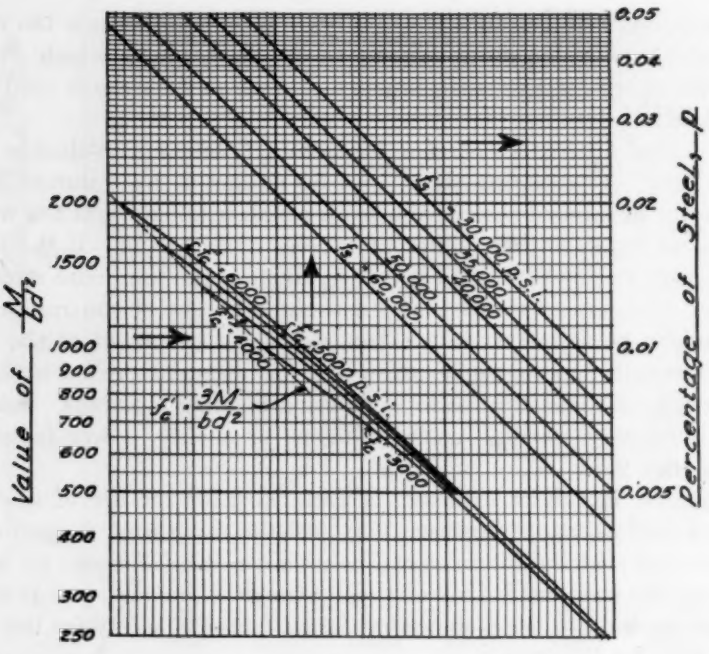


FIG. 21—ULTIMATE BENDING MOMENT AND PERCENTAGE OF STEEL FOR RECTANGULAR BEAMS—Based on yield point strength of steel, f_s and cylinder strength of concrete f_c'

factor of safety against compression failure than has a rectangular beam. This is probably particularly true when the concrete strength is high. The basic tests on the effect of T-beam flange proportions by Bach were made with very low strength concrete; and since the shear and tensile strength of concrete does not increase proportionately with the compressive strength, it is probable that T-beams of high strength concrete will not behave as well as those tested which failed by shear and diagonal tension in the flanges.

The writer believes that the adoption of the design method presented in this paper will result in more efficient concrete structures. During the last few years, since the development of the present standard portland cement and the resulting high strength concrete, he has observed an unusual amount of cracking of concrete slabs which presents a very serious problem. This is evidently due to the brittleness of the higher strength concrete which permits less plastic flow after setting and reduces the ability of the concrete to adjust itself to shrinkage strains. The more effective use of the concrete in com-

pression permitted by the proposed formulas will reduce the emphasis on high compressive strength and permit more leeway in the development of concrete with higher plastic flow and tensile strength which is badly needed.

DESCRIPTION OF TESTS PLOTTED IN FIGURE 19

The Humphrey and Losse tests (Technological Papers of the Bureau of Standards No. 2, 1912) were made on 333 8-inch by 11-inch beams of gravel, limestone, granite, and cinder concrete varying in strength from 1500 to 5700 p.s.i. Reinforcement varied from 0.49 to 1.96 per cent of steel with a yield point averaging about 40000 p.s.i. Eighty-four beams are not plotted because the concrete cylinder strength exceeded the testing machine capacity and f'_c was therefore not determined. The cylinders tested are 8 inch by 16 inch and f'_c, the strength of 6 inch by 12 inch cylinders, is calculated as $\frac{100}{95}$ times the reported value. Each point plotted represents the average of three beams.

The U. S. Geological Survey Tests (*Proceedings*, Amer. Concrete Inst., Vol. 16, 1920, p. 127) were made at St. Louis in 1908 on a series of 8 by 11 in. beams of gravel concrete with 3.02 per cent of steel with an average yield point strength of 38,210 p.s.i. Only beams with concrete strengths over 2000 p.s.i. are included and because individual steel strengths are not reported, only the averages are plotted for two groups, one under-reinforced (Beams 1116, 1131, 1132, 1133, 1134, 1135, 1136, 1137, 1140, 1141, and 1142) and one over-reinforced (Beams 1110, 1112, 1119, 1120, 1121, 1123, 1128, 1129, 1130, 1138, and 1139. The concrete cylinders were 8 by 16-in. and f'_c is calculated to be $\frac{100}{95}$ times the reported strengths.

The Columbia University tests (*Concrete and Constructional Engineering*, March 1937, page 200 and JOURNAL Amer. Concrete Inst. Nov.-Dec. 1936, *Proceedings* Vol. 33, p. 183) were made on thirteen rectangular 12 by 13-in. beams reinforced with ordinary steel and Isteg bars with yield points from 34,800 p.s.i. to 58,250 p.s.i. and areas varying from 0.451 to 1.245 per cent. The concrete strengths varied from 2510 to 3443 p.s.i. The yield point of the Isteg bars was taken as the stress producing a unit strain of 0.004.

The Slater and Lyse tests (JOURNAL Amer. Concrete Inst. Vol. 33, p. 488) are those previously quoted. The four sets excluded from the averages in Table I are not plotted. Each point represents the average of a group of three beams.

The Glanville tests (*Concrete and Construction Engineering*, March 1937, page 199), were made on 4 by 7 in. beams of concrete varying from 1505 to 7600 p.s.i. cube strength. The steel varied from 0.79 to 2.19 per cent with the yield point from 42,400 to 48,200 p.s.i. The value of f'_c is calculated from the six inch cube strength by means of the curve given in Fig. 20. This data does not conform accurately with the results of the other series and the poor agreement may be due to the small size of the beams and the uncertainty of the equivalent cylinder strength. All of the beams represented by the scattered points have concrete strengths of about 2000 p.s.i. or less so the disagreement is not very important.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . MARCH-APRIL 1937

Report of Committee 710

High Early Strength Cement in Concrete Masonry Manufacture*

BY BENJAMIN WILK, CHAIRMAN

At the 1936 convention this Committee presented a report‡ which was summarized as follows: "These tests indicate that for the method used (3 days moist room curing then air curing until test) 70 lb. of high early strength cement gave compressive strengths somewhat higher than 94 lb. of normal portland cement at ages of three to twenty-eight days. The difference in strength was more marked at the early ages (4 and 7 days) than at the 28 day period."

Tests under way last year to determine the effect of durability of the reduced cement content when using high early strength cement can now be reported. These tests on freezing and thawing, and on volume changes were made at Lehigh University. The volume changes, or shrinkages, are shown in Fig. 1. The figures show the shrinkages during approximately six months dry storage.

It is evident from these figures that there is practically no difference between the shrinkage of the high early strength cement concretes and the normal cement concretes. The aggregate, however, had a marked effect. The cinder concrete had a greater and the slag concrete considerably less shrinkage than the gravel concrete.

Comparative strength of high early strength and normal portland cement concretes subjected to 100 cycles of freezing and thawing are shown in Table 1. The test specimens, 3″ x 6″ cylinders, were made as were those described in the report of 1936.

*Presented at 33rd Annual Convention, New Yrok, Feb. 23-26, 1937.
‡JOURNAL, Amer. Concrete Inst., May-June, 1936, *Proceedings* Vol. 32, p. 673.

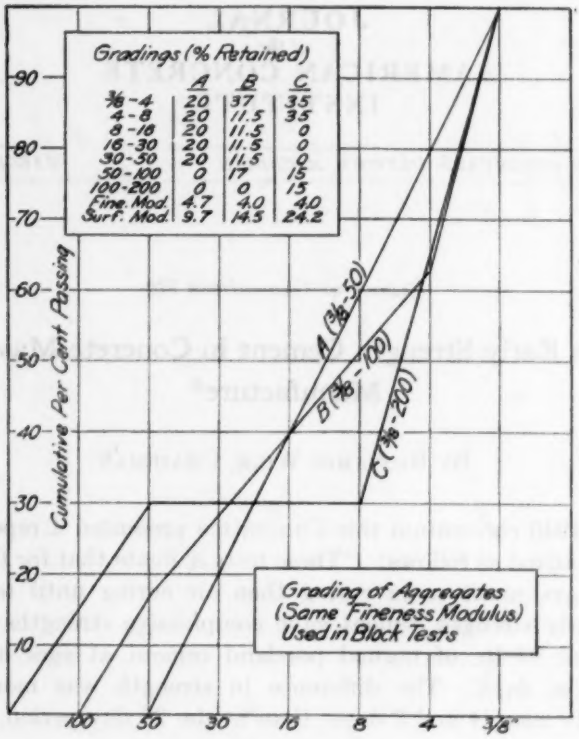


FIG. 1

TABLE 1—COMPARATIVE STRENGTHS OF HIGH EARLY STRENGTH AND NORMAL PORTLAND CEMENTS SUBJECTED TO 100 CYCLES FREEZING AND THAWING

Test specimens 3 by 6-in. cylinders.
Specimens made at 70° F. and cured 3 days moist at 70° F. followed by storage in air of laboratory until time for test.
Fineness modulus of aggregate 3.8.
Seventy pounds of high early strength cement per batch compared with 94 lb. normal portland cement per batch.
Specimens made and tested at Lehigh University.

Cement	Compressive Strength—p.s.i. Days of Moist Curing Before Exposure to Test								
	Cinder Concrete			Gravel Concrete			Slag Concrete		
	3D	7D	28D	3D	7D	28D	3D	7D	28D
Normal Portland									
N1.....	2230	2450	2630	4200	4470	4210	4000	4290	3920
N2	60 cyl.	110 cyl.	2300	4040	4950	4380	4070	4550	4350
Average...			2465	4120	4710	4295	4035	4420	4135
High Early Strength									
H1.....	1830	1700	2000	4110	4720	5480	4180	4300	4470
H2.....	5 cyl.	2120	2310	4200	4670	3770	3000	3950	4390
Average...		1910	2155	4155	4695	4625	3590	4125	4430

It is evident from this table that, except for the cinder concrete specimens, some of which were not able to withstand 100 cycles of freezing and thawing, the concretes withstood the tests very well. A comparison of the tests in this table, with tests on similar cylinders tested before freezing and thawing—reported last year—shows that the strengths of the specimens tested after, exceeded those of specimens tested before freezing and thawing.

Considering all of the results of freezing and thawing tests it is evident that the reduced cement content in high early strength cement specimens does not reduce the resistance of the specimens to freezing and thawing.

Last year the Committee's tests on the effect of grading showed it was of advantage to use fine material passing the 100 mesh in block mixtures. Inasmuch as these finer gradings resulted in the lowering of the fineness modulus, the Committee decided to investigate the effect of different gradings having the same fineness modulus. Three gradings were made: 3/8-50, 3/8-100, 3/8-200, each with the fineness modulus of 4.00 with surface moduli of 9.7, 14.5 and 24.2 respectively. (Fig. 2 and Table 2.)

TABLE 2—GRADINGS OF AGGREGATES

Sieve	Per Cent Passing Sieve Sizes Indicated		
	⅜—50	⅜—100	⅜—200
⅜"	100	100	100
No. 4	80	63	65
8	60	51⅓	30
16	40	40	30
30	20	28⅓	30
50	0	17	30
100	0	0	15
F. M.	4.0	4.0	4.0
S. M.	9.7	14.5	24.2

Cylinders, were made as in the previous tests, at the Central Laboratory of Lehigh Portland Cement Co., except that sand and gravel only were used as aggregates. The batches were designed to give a yield of one 8 x 8 x 16 building unit for each 3.56 lb. of high early strength cement and one unit for each 4.75 lb. of normal cement. The cement content per batch as used varied slightly from the designed batches. The program involved the following variables:

2 cements, normal portland and high early strength portland
1 aggregate; sand and gravel
3 gradings, as indicated in Table 2
3 ages at test; 4, 7 and 28 days
1 curing; 3 days in damp room then in laboratory air
3 companion specimens at each age test in room dry condition.

Table 3 shows the effect of the variable gradings on compressive strengths and the percentage increase of the high early strength cement concretes over the normal cement concretes.

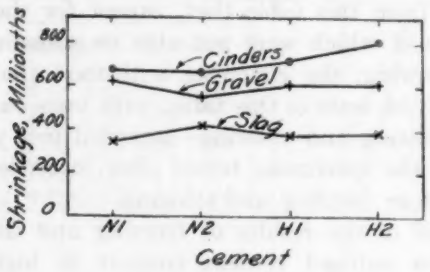


FIG. 2

TABLE 3—EFFECT OF GRADING ON COMPRESSIVE STRENGTH

Age Days	⅛—50 Agg.			⅛—100 Agg.			⅛—200 Agg.		
	Cement		% Increase	Cement		% Increase	Cement		% Increase
	Normal	H.E.S.		Normal	H.E.S.		Normal	H.E.S.	
4	1490	1700	1.14	2150	2640	1.23	2150	2680	1.25
7	2190	2350	1.07	3020	3350	1.11	2860	3190	1.12
28	2540	2560	1.01	3740	4110	1.110	3600	3780	1.05

The following conclusions are indicated from this group of tests:

1. The mixtures are distinctly benefitted by fine aggregate below the 50 sieve while the second grading gives slightly higher results than the third grading. There is a suggestion that the best grading lies between these two—about 5 per cent through the 100 mesh.

2. On all gradings and at all ages the high early strength cement shows, however, more efficiency and economy than the normal cement.

Compressive tests have also been conducted on concrete blocks made at intervals at the plant of the Standard Building Products Co. in Detroit where high early strength cement has been used in large quantities and almost exclusively during the last year. These tests have corroborated the results reported by the Committee last year.

Taking into account the results reported this year, as well as last year, it would seem that on an average 70 lb. of high early strength cement will give better results than 94 lb. of normal portland cement during the first 28 days in the manufacture of concrete masonry units. This takes into account—strength, volume change and resistance to freezing and thawing.

The Committee suggests that high early strength cement should not be used indiscriminately, that product manufacturers should make their own comparisons.

Discussion of the foregoing report will be welcomed if received in triplicate by the Secretary of the Institute by June 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . MARCH-APRIL 1937

Report of Committee 501—Standard Building Code

Building Regulations for Reinforced Concrete*

A. W. STEPHENS, CHAIRMAN
R. R. ZIPPRODT, SECRETARY

The By-Laws of the Institute require that amendments to a Tentative Standard shall be printed 120 days prior to adoption by the Institute. Committee 501 does not, therefore, present any matter for action at this Convention.

The Committee proposes to present, early this Spring, its recommendations for the amendment of Section 709, "Slabs Supported on Four Sides," for the purpose of simplifying the design procedure.

The Committee also expects to present its further recommendations for an amendment to Chapter 11, "Reinforced Concrete Columns."

The sub-committee of Committee 501 having responsibility for the review of Chapter 10, entitled "Flat Slabs," is actively engaged in its study.

In adopting the Committee's report as a Tentative Standard in February, 1936, the Convention of the Institute changed the provisions of Chapter 11—Reinforced Concrete Columns, as follows: Permissible load on the concrete section of a spirally reinforced concrete column from .25 f_c' to .22f_c', and the permissible load on tied columns from 80 per cent to 70 per cent of the total load permitted on spirally reinforced concrete columns.

In view of the recommendations which were made in the Progress Report of the new Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, and of recent discussions of the members of Committee 501, the belief is warranted that the Com-

*Presented at 33rd Annual Convention, New York, Feb. 23-26, 1937.

mittee will recommend shortly that the permissible load on the concrete section of a spirally reinforced concrete column be changed from $.22f_c'$ to $.225f_c'$, and that the permissible load on tied columns be changed from 70 per cent to 80 per cent of the total load permitted on spirally reinforced concrete columns.

Professor Sutherland, chairman of the sub-committee on Slabs Supported on Four Sides will present an outline of the revision of the textual content of Section 709, which will be proposed this Spring, as an amendment to the Tentative Standard (A. C. I. 501-36-T).

Discussion of the foregoing report will be welcomed if received in triplicate by the Secretary of the Institute by June 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . MARCH-APRIL 1937

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers.

Innovations in the burning of cement

H. JORDAN, *Zement*, Vol. 25, No. 46, Nov. 5, 1936, p. 773-777. Reviewed by L. T. BROWNMILLER

The more important German patents of 1935 relating to mechanical devices applicable to the kilns, etc., for aid in the burning process are reviewed. The descriptions are very brief and give only the merest outline of the patent.

Steam heating plant warms concrete aggregates

Engineering News-Record, Vol. 118, No. 7, Feb. 18, 1937, p. 273. Reviewed by R. W. BEAL

Concrete tunnel lining in the Mono Craters tunnel is being carried on by the City of Los Angeles when temperatures outside the tunnel are 20° below zero. A steam heating system was installed over the grates through which the aggregate moves from the stockpile to the mixing plant near the tunnel portal. The temperature of the mixed concrete is 60°, and it is easy to maintain this temperature after the mix enters the tunnel.

The kiln ring in theory and practice

T. YOSHII, *Zement*, Vol. 25, No. 42, Oct. 15, 1936, p. 727-732. Reviewed by L. T. BROWNMILLER

A study was made of the causes of ring formation, among which are enumerated: (1) the effect of the coal and ash on ring formation; (2) the effect of the construction of the kiln; (3) the effect of the kiln lining; (4) the effect of chemical composition of the raw mix; (5) the effect of the regulation of the kiln (air, rate of feed, etc.). Thus the ring may be caused by a combination of these factors and not by any single one. The methods of removal of the ring are discussed and some new improvements for removing it suggested.

Studies of the effect of 9 cements on the shrinkage, elasticity and strength of concrete for roads

FRITZ WEISE, *Zement*, Vol. 26, No. 3, Jan. 21, 1937, p. 39-43. Reviewed by L. T. BROWNMILLER

This paper contains principally the data up to 2 years on the properties of the cements and concrete made from them with no attempt to correlate the properties of the concretes with composition, etc. The data may be of some value for comparative purposes since there are 7 slag cements in the group, one portland and one high strength portland cement. Among the slag cements there is a considerable variation in manganese content from 0.43 per cent Mn_3O_4 to 2.42 per cent Mn_2O_3.

Hot cement

L. KRUGER, *Zement*, Vol. 25, No. 43, Oct. 22, 1936, p. 741-743 Reviewed by L. T. BROWNMILLER

Freshly ground cement is generally at a relatively high temperature as a result of the grinding of the clinker. Often it is stored in silos at the elevated temperature and the heat may not be dissipated for a long time. A series of tests is reported on the strength, the setting characteristics, and the volume constancy of hot cement. In making the tests the mixing water was preheated to temperatures as high as 30° C. At the higher temperatures the time of set was decreased. The other properties of the cement were not affected adversely by using freshly ground hot cement (70° C.).

Changed concreting practice speeds road widening

A. T. BECK, *Engineering News-Record*, Vol. 118, No. 9, Mar. 11, 1937, pp. 367-368.
Reviewed by R. W. BEAL

Reconstruction work on main highways in Wisconsin has led to the conclusion that it is economical to salvage the old pavement only when minor corrections in alignment or grade are to be made, or when additional width is the primary requirement. Time and money are saved and the finished road is more satisfactory, if the widening is placed at the same time and monolithically with the resurface course. When a new surface is placed over old pavement, it has been found necessary to place the new expansion joints over the old joints to prevent cracking of the new surface.

Self-supporting reinforcing in concrete bridges

Engineering News-Record, Vol. 118, No. 7, Feb. 18, 1937, p. 270-271. Reviewed by R. W. BEAL

In the construction of the 90 ft. high approach to the highway bridge across the Fraser River at New Westminister, B. C., light steel trusses were set in each span to serve a dual purpose: during construction they support the dead weight of the concrete and forms, and after the concrete is set they form the reinforcing within the concrete girders. The steel trusses were erected on the concrete substructure by a traveling derrick, making the usual falsework unnecessary, and each truss was given sufficient initial camber to allow for the expected deflection under the load it would carry while the concrete was setting. The cost of the entire approach structure was about $230 per lin. ft.

Large sewer on ocean bottom has joints of novel design

Engineering News-Record, Vol. 118, No. 9, Mar. 11, 1937, pp. 369-373. Reviewed by R. W. BEAL

A 5000-ft. outfall sewer of 5-ft. internal diameter is under construction by the Los Angeles County sanitation districts. The concrete pipe is made up in 12-ft. and 18-ft. sections with cast iron joints which are welded to the ordinary concrete pipe reinforcing cage. Where no joint articulation is necessary, the rigid, inside-calked Lock-Joint type of joint is used; while outside-calked, ball-and-socket type joints are used when flexibility is required. The former type of joint can be made up under water by a diver in four hours, but the latter type requires about 44 hours. The design is believed to be proof against joint leakage and breakage such as has occurred in other ocean outfalls of large diameter where all-concrete pipe has been used.

Retaining wall counterforts support concrete arch roof

Engineering News-Record, Vol. 118, No. 8, Feb. 25, 1937, pp. 291-292. Reviewed by R. W. BEAL

High early-strength cement was used in cold weather in the construction of a reinforced-concrete parabolic arch roof by the Lone Star Cement Corp. at its Nazar-

eth, Pa., plant. The roof has a 79-ft. span, a 16-ft. rise, is 6 in. thick at the crown and 12 in. at the springing line, and is supported by extensions of the counterforts used in the side walls. At the column supports the girders are tied across the building by two 2¼-in. steel rods.

A thin layer of 4-in. slump concrete was deposited first so that the bottom reinforcing would be fully embedded and undesirable honeycombing eliminated. The remaining concrete had from 1 in. to 2½ in. slump, depending upon the steepness of the arch where it was to be placed. All forms were removed within 48 hours.

Curing concrete in desert heat

Contractors and Engineers Monthly, Nov. 1936, p. 27. HIGHWAY RESEARCH ABSTRACT

In constructing the Fan Hill siphon, on the Colorado Aqueduct, several methods of air fogging were tried within the finished transition sections with a view to securing the best results with the use of a minimum amount of water. Numerous small jets were threaded into a 1-inch pipe line, which was suspended 2 feet below the ceiling of the finished sections after they were bulkheaded off with canvas to keep out air currents. With a water pressure of 80 to 87 pounds, the jets were opened on 2-foot centers and run constantly for the first 72 hours. For the next four days the jets were opened on 4-foot centers constantly. Then for 48 days the jets were opened on 6-foot centers, with water turned on 10 minutes and off 30 minutes. The spray hit the ceiling, filled the air, and trickled down the sides, maintaining a humidity of 95 to 97 per cent.

Economics of highway-bridge floorings of various unit weights

J. A. L. WADDELL, *Proc. Am. Soc. C. E.*, Vol. 63, No. 2, p. 301. Reviewed by H. J. GILKEY

Using a 110-lb. per sq. ft. reinforced concrete floor, the usual type, as a basis for comparison, relative costs are plotted against span lengths for a variety of bridge types and flooring types. The bridge types include highway bridges, both simple truss and cantilever or bascule, of both carbon and silicon steel, and of roadway widths of 45 and 20 ft. There are eight floor types ranging from 123 to 15 lb. per sq. ft., the lightest being an open grating. In all cases, the lighter weights offer the greatest saving in cost for all spans of bridge. The eminence of the author as an authority on bridges places the stamp of authority upon the essential validity of the conclusions reached. The author asserts that these conclusions could be affected but slightly by changes in relative costs of materials. The paper supplies essential background information on the conditions assumed.

Foaming agents for light-weight concrete

N. JERMALENKO and N. ABRAMTSCHOK, *Zement*, Vol. 25, No. 51, Dec. 17, 1936, p. 882-885.
Reviewed by L. T. BROWNMILLER

In making light-weight concretes, generally air or gas is entrapped in the wet concrete which then sets and consists of a porous tuff-like mass. Authors have studied a series of liquids which form foams of some stability, mixed them with cement, allowed the cement to set and in that way made light-weight concretes. The success of such a process depends largely on the stability of the foam and its effect on the hardening of the concrete. They report here a series of studies of a preliminary nature on the types of foams they have found most suitable for this particular use and the result on the 7 and 28 day strength of light-weight concrete made with the foams. The data included in the article certainly should be valuable to anyone contemplating entering this field, either for further study of the problem or for the actual manufacture of light-weight concrete. The foam-producing materials reported on are waste products of animal or vegetable nature.

New spun concrete pipe and flint-lime brick plant

Concrete Building & Concrete Products, Vol. 12, No. 1, Jan. 1937, pp. 5-10.

Reviewed by J. C. PEARSON

This article describes a million-dollar plant at Cowley, England, thoroughly equipped to produce only the two remotely related products indicated in the title. A rather remarkable story is told of the use of flint-lime bricks of this type in one of the Simplon tunnels, where they replaced granite block liners which were failing. These bricks are made of a 3:1 mixture of hand crushed gravel and lime, steamed and molded under pressures as high as 16000 p.s.i. From the presses the bricks are trucked to the autoclaves where they are cured under high pressure steam for 8 hours and are then ready for delivery. The autoclaves are 10 in number, each 68 ft. long and having a capacity of 16,500 bricks. The daily capacity is 200,000.

The concrete pipes are made on a centrifugal machine having unusual features. Uniform thickness of walls is assured by a system of carefully controlled unit batches, and by a scheme of vibration which spreads and compacts the concrete prior to spinning. The walls of the mold are perforated and lined with strong linen which acts as a filter. Under centrifugal force the excess water is thrown outward, and only a thin film of neat cement is left on the bore of the pipe. A test of one of the lightest pressure pipes, which was 12 ft. long, 15 in. bore and 1¾-in. thick showed no seepage up to appearance of the first crack, which occurred at 130 p.s.i.

Experiments for the development of road cements

H. E. SCHWIETE, *Zement*, Vol. 25, No. 46, Nov. 12, 1936, p. 791-798.

Reviewed by L. T. BROWNMILLER

The development of improved cements for road construction may be done by changing the chemical and physical properties of the normal cements or by the development of new special cements. By changing the chemical composition of normal cements, the mineral constituents may be varied considerably. This variation brings about changes in properties so that by proper selection, the best cements for special purposes may be chosen. For highways, the tricalcium silicate content is of foremost importance. From tests the author made on some experimental cements, increase in $3CaO.SiO_2$ generally increased strength and decreased shrinkage. Author intimates that a high $3CaO.SiO_2$ cement is preferable for road construction; however, the optimum amount of $3CaO.SiO_2$ must be determined for each plant and each raw mix. These conclusions were based on 28 day results which represent rather a short time test.

Fineness is also important in evaluating quality for highway cements. A decrease in specific surface improves the elastic properties of the cement and decreases shrinkage. The optimum fineness must be determined from the allowable loss in strength due to the coarser cement and from the poorer workability of coarse cement mixes.

None of the special cements investigated showed properties which were equivalent to those of normal cements according to the methods of testing.

Vibrated concrete and vibrators in practice

HOLLENSLEBEN, *Beton u. Eisen*, Vol. XXXVI, No. 3, Feb. 5, 1937, p. 41.

Reviewed by A. U. TREUER

The author presents a general discussion of vibrating methods and machinery in their application to concrete placement. After listing several outstanding structures in which vibration methods of placement were used, the author discusses field proportioning and illustrates the economy effected by the use of vibration methods of placement. The use of a 1:2½:4½ mix for conventional methods of placing as

against a 1:2½:7 mix for vibration methods is cited as being typical of field practice for plain concrete. The saving in cement is roughly 15 per cent.

The success of vibration methods is found dependent, to a high degree, on the choice of proper equipment. Of fundamental importance is the choice between air-pressure and electric vibrators. The former is characterized by sharp hard jolts or blows and the latter by a semi-harmonic or stirring motion. The air pressure type is found more effective in consolidating a mass of concrete but more expensive in cost of operation. Data are included giving values of the power consumed by air pressure vibrators of varying weights.

The three methods of vibrating (forms, surface, internal) are discussed with reference to their special applications. The proper frequency of vibrators is cited as 3600 r.p.m. Higher frequencies, even if more effective, are considered as questionable, economically, with present day equipment, because of increased service costs. The effects of varying amplitude and force are discussed briefly.

Degree of burning and clinker quality

G. Muszgnug, *Zement*, Vol. 25, No. 1, Jan. 7, 1937, 1-7. Reviewed by L. T. Brownmiller

Cements of approximately the same composition often show considerable difference in properties which cannot be explained on the basis of composition alone, but which may be caused by differences in degree of burning or variations in cooling. The studies reported here are chiefly on the effect of degree of burning of the clinker. The clinkers were divided into three groups depending on the degree of burning,—that is the temperature of burning. Even the clinker burned at the lowest temperature was not an underburned clinker, but could be classified as a normal cement clinker. The raw mix contained a blast furnace slag with appreciable sulphate. The SO_3 content of the clinker was taken as a measure of the degree of burning. Other variables were eliminated in so far as possible. However grinding was not to a uniform surface area as might have been desirable; but sieve analyses were relied upon to express fineness. The effect of the degree of burning on the following properties were studied: chemical composition, alkalies, specific gravity, unit weight, setting time, volume constancy, strength, water requirements, shrinkage, heat of set and microscopic structure. Two series of tests were made, one in which 3 per cent gypsum was added to all clinker and the other in which gypsum was added but in varying amounts so that the total SO_3 was constant for all clinker. The two series showed appreciable differences in properties because of this difference in sulphate content. In general the overburned or sharply burned clinker had poorer properties than the lower burned, normal clinker.

The distribution of loads on floor beams

Hjalmar Granholm, *Betong*, Journal of the Swedish Concrete Institute, No. 4, 1936.
Reviewed by Inge Lyse

In the design of bridge floors consideration must be given to the distribution of the load on the different floor beams and girders. The author presents a most complete and competent analysis of this problem. He first considers the general case of three floor beams of variable moment of inertia and variable stiffness of floor plate under uniform load. The fourth differential of the deflection curve for each beam gives values proportional to the load carried by each beam and the author proceeds to develop the general equations. Special cases are then considered, such as beams of constant moment of inertia and floor plates of uniform thickness, simply supported beams, and beams with fixed ends as well as continuous beams of constant stiffness. Examples are worked through for the illustration of the practical applica-

tion of the theoretical analysis. One chapter is devoted to the solution of the load distribution equations by means of Fourier's series for the different conditions of end restraints and for the case when cross beams are used. Another chapter considers the load distribution between beams of high torsional restraint and the effect of the plate action when the supporting beams have high torsional rigidity. The theoretical analysis and a simplified practical application are presented. The paper contains 81 pages with 17 illustrations and is a most welcome addition to our very scant literature on the complexity of an accurate analysis of the actual behavior of reinforced concrete structures where floor slabs, beams, girders, columns and walls all act as a monolithic unit.

Eccentrically loaded columns or walls

N. ROYEN, *Teknisk Tidskrift*, Vol. 67, No. 4, Jan. 23, 1937 Reviewed by INGE LYSE

Consideration is given to columns and walls of materials which have little tensile strength, such as bricks and plain concrete. Thin reinforced concrete walls with reinforcement only in the center are also considered. Theoretical studies are made of the action of these structures when a load is applied eccentrically. Three different conditions are analyzed, namely:

1. The load is applied within the kern section throughout the column.
2. The load is applied outside of the kern section throughout the column.
3. The load is within the kern section near the ends but otherwise outside this section of the column.

Stress formulas are developed for each of these different conditions and the results presented graphically. The critical loads for conditions 2 and 3 are found to be nearly equal and may be represented by the formula:

$$P_{critical} = 76\left(\frac{1}{2} - \frac{e}{h}\right)^3 \frac{EI}{l^2}$$

where $P_{critical}$ is the critical load, e is the eccentricity, h the total width of the section E the modulus of elasticity, I the section modulus, and l the length of the column,

The analysis of the columns and walls with reinforcement at the center only gave a critical load approximately equal to that given in the above formula when the ratio $\frac{e}{h}$ was less than 0.25; when $\frac{e}{h}$ became greater than 0.25 the critical load became:

$$P_{critical} = 1.22\,\frac{E \cdot I}{l^2}$$

Autogenous healing of concrete

(1) M. W. LOVING, *Bull. 13, American Concrete Pipe Association*, 1936, 8 p.; (2) V. J. SOROKER and A. J. DESOV, *Zement*, 25 (30) 1936, p. 505-6. Abstract from Road Abstracts, Jan., 1937, No. 579. HIGHWAY RESEARCH ABSTRACT

1. In the state of Washington in 1929-30, a number of culverts were built with extra-strength reinforced concrete culvert pipe under fills from 40 to 70 ft. and on grades up to 34.8 per cent. These pipe lines were examined in August, 1931 and all defects recorded. Re-examination was made in August, 1936. In 1931, the pipes showed cracks which measured from hair to .03 in., but by 1936 these had been sealed by autogenous healing. Considering the enormous initial stresses to which these pipe lines were subjected, such fine cracks could naturally be expected, but it is revealing to have found that nature, through autogenous healing, has developed practically perfect culverts. In the presence of moisture, concrete exudes calcium hydroxide which, upon exposure to the atmosphere, is converted to calcium carbon-

ate. This seals the cracks and prevents oxidization (rusting) of the metal reinforcement.

2. Eight-inch concrete cubes were made up in various proportionings and at the age of 8 days were subjected to measured compression until the formation of cracks. They were then stored at a temperature of 15° to 19° C., with a relative humidity between 85 and 90 per cent for 30 days or 60 days, and their crushing strengths at those ages compared with the initial values obtained. The results showed very marked increases in final strength values. In a second series of tests cubes were stressed to cracking at the age of 8 and 30 days, were stored as before for 60 days, and then broken in compression. The final strength values were markedly higher, as compared with the 8-day cracking strength, than with the 30-day cracking strength. In both series of tests the final crushing strength increased with increase in the initial cracking strengths.

The control of the moisture content of aggregates for concrete, introducing a new vibration method

Road Research Technical Paper No. 4, Department of Scientific and Industrial Research and Ministry of Transport, England. AUTHOR'S ABSTRACT

A new method for ensuring greater uniformity in the strength and other properties of concrete used for road making or building construction has been devised by the Building Research Station of the Department of Scientific and Industrial research and developed at the Department's Road Research Laboratory, England.

The method consists in saturating the materials used in making the concrete and then vibrating them for a given length of time so as to give them a constant water content less than that required for complete saturation. In the actual machine used the container is attached to a steel plate which is shaken by eccentric weights keyed to the shaft of a petrol engine running at 1500 r.p.m. When running at this speed water is turned on and sprayed into the container by means of a hose and the sand and gravel from the stock pile is shoveled into the container. When this is overflowing the water is turned off and the sand vibrated for, say, 30 seconds. The material is then discharged into the hopper or other mixing plant. In practice, time taken to load, vibrate and discharge the sand is 90 seconds but this time it is suggested could be reduced with more extensive vibration at a higher frequency.

The results described show that a very uniform sand and water mixture is produced by this method. Tables indicate how the actual percentage of moisture content in the sand is affected by the frequency in the amplitude of the vibration, the time of vibration and also the grading of the sand.

In practice the vibrator can be used in one of several ways. It can be operated by the mixer engine and fixed to the mixer or it can be made into a portable self contained unit or again, can be used to condition stock piles of materials.

In tests described in the report the use of vibrated material raised the minimum concrete strength from 2,300 to 4,500 p.s.i. The average figure was consequently considerably increased and the variation from this average was reduced from 23 to 10 to 11 per cent.

Special cements for mass-concrete structures and their specifications

Report to the Joint Sub-Committee on Special Cements of the Institution Research Committee and the British Committee on Large Dams. F. M. LEA, D. Sc., F. I. C. Reviewed by R. N. YOUNG

As a representative of the Sub-Committee at the Second International Congress on Large Dams which was held at Washington, D. C. in September, 1936, the author reports to the Sub-Committee regarding the progress made in the manufacture of

cement having lower heat of hydration, their present status and the specifications and properties of such cements. Particular attention is called to the tendency in the U. S. A. to depart radically from usual specification methods. One function of the Sub-Committee was to indicate to the manufacturers the properties of the cement desired and the tests by which these properties are to be evaluated, rather than to describe how the cement should be manufactured. Their attention therefore was confined to consideration of heat of hydration, drying-shrinkage, permeability and workability of concretes. The thought was stressed that special cements may be expected to overcome certain troubles in mass-concrete construction but that they would in no way eliminate the necessity of using every precaution to obtain high quality concrete.

The American practice of emphasizing the fineness and chemical composition in specifications implies that if the composition and fineness of portland cements are specified, the properties of the cements directly follow, and that this forms the basis for recent U. S. specifications for special cements. As a further contrast between the European and American forms of specifications the latter places the responsibility of the behavior of the cement when used, upon the engineer who formulates the specification. Moreover the strong dependence on fineness and chemical composition requirements is an attractive procedure in that the methods are rapid and accurate.

Sweden is the only country other than the United States that has had appreciable experience with the use of low-heat cement, the development of which was contemporary with that in this country. The Swedish low-heat portland cement is not ground as fine as the American cement in order to obtain slower setting which permits the placing of larger masses of concrete. Poor workability was encountered and this lead to the addition of 3 per cent of diatomaceous earth (by weight of cement) to the concrete mix. Their adoption of a low fineness requirement was influenced by the construction methods employed. For low-heat cement, including puzzolanic cement, the Swedish method of specification is to specify heat evolution and strength but not to use detailed requirements for chemical composition or cement ingredients.

Much of the contents of the report by Mr. Lea, that is not referred to here, is in the nature of a review of the Report on Special Cements presented to the Second International Congress on Large Dams, at Washington, D. C. by J. L. Savage, Chief Designing Engineer to the U. S. Bureau of Reclamation.

Grouting construction joints at Boulder Dam

JAMES B. HAYS, *Civil Engineering*, Feb. 1937, Vol. 7, No. 2, p. 126. Reviewed by J. R. SHANK

This paper (fourth on this reviewed in these pages recently) is an abstract of a much longer one which gives the details of the grouting work at Boulder Dam and is on file in the Engineering Societies Library.

The contraction joints were divided into two classifications, radial joints and circumferential joints, and were grouted separately. The radial joints were grouted first, at least one week ahead of the circumferential joints.

The general procedure was to fill the joints with water and allow to soak for several hours. The grout was them pumped in driving the water ahead of it out through outlet valves. After the joints were filled, pressure was applied to drive excess water into the concrete, thus consolidating the grout. Dial gages were used at the joints to measure the possible joint opening. Cement through a 200 mesh screen was generally used because held more readily in suspension with less wear on the equipment.

Central mixing plants and group groutings were used. Several joints in a region were grouted more or less simultaneously. The application was changed periodically

from one to another bringing all finally to completion. This kept the blocks better balanced against lateral movement. There was care to avoid placing the central mixing plant too high above the joints being grouted. The limiting elevation was that at which the gravity pressure of the grout did not exceed the maximum working pressure desired. The total area of a group of joints was generally limited to 100,000 sq. ft. An average speed of more than 200 sacks of cement per hour was maintained for a full shift. The maximum was 240 sacks. Circumferential joints took approximately one-fourth less grout than the radial ones. The volumetric water-cement ratio varied from 1.00 to 0.70, thicker grouts did not distribute pressure evenly.

With all joints full, a light trial pressure of about one-half the final pressure was first given to each joint in turn to re-test the outlet headers for trapped air. The limit of pressure was generally 50 lb. When a dial gage across a joint indicated movement, this pressure was reduced.

One and five-eighths inch core borings revealed the following:

1. The bottom of several of the joints were filled with sediment, sand, and laitance.
2. Some of the tops were incompletely filled or contained a chalky material, probably laitance from the grout.
3. Practically all joints showed a thin white layer between the grout and the concrete—a coating left on the concrete by the curing and wash water.
4. Double layers of grout were found in joints which had leaked during the grouting of the foundation when pressures ranging up to 900 lb. were used.
5. Where dial gages placed across the joints at the bottom of a lift indicated a wide opening during the applications of pressure, the cores were never found to be held together by the grout.
6. Joints thought to be incompletely grouted were found well filled.
7. Joints as thin as 0.01 inch were found well grouted.

Gasoline consumption, tire wear, and coefficients of friction on various road surfaces

R. A. Moyer, and H. W. Tillafaugh, (Iowa Engineering Experiment Station) *Proceedings of the Highway Research Board*, Vol. 16. Highway Research Abstract

During 1936 the Iowa Engineering Experiment Station conducted a series of tests in which an attempt was made to correlate gasoline consumption, tire wear and coefficients of friction for five representative types of road surfaces. Such information is intended to be of assistance in determining transportation costs, and accordingly in selecting the surface which will provide transportation at the lowest cost consistent with safety.

Test surfaces included concrete, rock asphalt, penetration macadam, oiled gravel and untreated gravel. While this paper is in the nature of a progress report, following are some of the conclusions which appear to be warannted:

1. Gasoline consumption tests in traffic over long test courses provide no information which cannot be obtained in carefully conducted tests over one or two-mile courses at considerable saving.
2. The best gasoline mileage was obtained on the portland cement concrete surface, followed closely by rock asphalt, penetration macadam, and oiled gravel. Untreated gravel showed a gasoline mileage approximately 10 per cent lower than the portland cement concrete.
4. The relative change in gasoline mileage caused by operating at 33 and 52 miles per hour remained fairly constant for all surfaces, with about a 22 per cent reduction in consumption at the higher speed. Variations due to speed far outweigh those caused by differences in road surfaces.

4. Oil consumption at 52 miles per hour was about five times greater than at 33 miles per hour.

5. Tire wear on portland cement concrete and sandstone rock asphalt, were about the same at 33 miles per hour, and also at 52 miles per hour.

6. The greatest tire wear was obtained on the untreated gravel, the wear at 33 miles per hour being 2.7 times as great as on the concrete at this speed.

7. The wear on the untreated gravel remained practically unchanged at 33 and at 52 miles per hour.

8. The wear on the rear tires was double that on the front tires on hard surfaces, but remained the same on the untreated gravel at 33 miles per hour. At 52 miles an hour the wear for the rear tires on all surfaces was about double that obtained on the front tires.

9. The only punctures encountered in 16,300 miles of tests were on 2,000 miles of untreated gravel, where two punctures occurred.

10. Tire wear on the right side was 30 per cent greater than that on the left side, due largely to the effect of crown.

11. Total wear for all tests showed that the least wear was on the left front wheel, while the greatest wear, on the right rear wheel, was three times as much.

12. In correlating the results of the skid tests and tire wear test, tire wear on oiled gravel, which had the lowest friction coefficient, was one-half to one-third as great as that on portland cement concrete; rock asphalt, with the highest coefficient, showed wear only slightly greater than that on the portland cement concrete; the "non-skid" penetration macadam with coefficients about the same as portland cement concrete provided average wear slightly lower than that obtained on portland cement concrete; the untreated gravel with low coefficients, especially in the side-skid tests, provided the greatest tire wear of all the surfaces tested.

Mount Bold reservoir dam on the River Onkaparinga, South Australia

HERBERT HENRY CARTLEDGE (writing on Construction); CHRISTOPHER GEORGE FREDERICK JOHNSON and ARCHIBALD HECTOR CAMPBELL (Concrete Control) *Journal of the Institution of Engineers*, Australia, Vol. 9, No. 1, Jan. 1937, p. 1 to 23. Reviewed by J. R. SHANK

This dam, about 22 miles from Adelaide, is an impounding or regulating dam for the Happy Valley Reservoir. It is an arch type concrete dam containing 132000 cu. yd. of concrete and was constructed at a cost of £480,000.

The aggregate used, typical for Australia, was a crushed rock made up largely of quartzite which contained a variable amount of sandstone. The fine aggregate, as well as two sizes of coarse aggregate, was made up of this material. The concrete was mixed very dry, to no slump and placed by means of 8-in. square tampers on jackhammers. The handling was done by a 10-ton Ligerwood type cable-way with a fixed tail tower and a movable head tower having a cable length of approximately 1100 ft. Electrical power was used. Three main mixers each of 1 cu. yd. capacity and one ½ cu. yd. mixer in reserve did the mixing. Bottom dump buckets which opened on two opposite edges were used. The pile of green concrete was spread out to a half-foot thickness and rammed into place. Before concrete was placed on a previous pour, all laitance was removed to a depth of one-quarter inch by means of four point power-driven drill bits mounted on a large wheelbarrow which was moved slowly over the whole surface. The lining of an outlet tunnel was so heavily reinforced that it became necessary to place concrete by means of grocer's sugar scoops passed through the reinforcing bars to men squeezed inbetween the layers of steel. This lining concrete notwithstanding, was mixed and placed for about 17s. 6d per

cu. yd. inclusive of all plant costs. An interesting feature of this work was the use of recording ammeters on different parts of the work for obtaining an automatic daily record of performance. The chart from the cable-way ammeter recorded the number of buckets placed as well as all delays. The effect was to speed up the work.

The concrete was proportioned by weight to 1:2.25:1.90:3.50. The volumetric water-cement ratio was 0.95 to 1.00. It had a compressive strength of 3800 p.s.i. at 34 days and a weight per cu. ft. of about 145 pounds. The physical properties of the aggregates were:

	Sand	Medium Aggregate	Coarse Aggregate	
Weight per cu. ft.	86 to 95	80 to 84	80 to 82	as batched, average
Fineness Modulus	2.71	6.67	8.14	one report
Size range	200 to ¼″	No. 4 to ¾″	¾″ to 2½″	one report
Specific Gravity	2.58	2.51	2.53	one report
Voids, per cent	40.5	47.5	49.5	one report

Some interesting things learned concerning this aggregate and concrete situation are as follows:

1. The water cement ratio had to be held in a very close range, from 0.96 to 0.99 to obtain uniform results.
2. Mixing time is not economical under 2½ minutes and may be continued with increased strength and workability up to 10 minutes. Times longer than this showed decreasing strength. A 4-minute mix is advisable.
3. The per cent sandstone or other inferior material content had a marked effect on the strength: 3400 p.s.i. at 9 per cent to 4200 p.s.i. at 3 per cent.
4. Core drillings 1.4 in. dia. x 2.8 in. showed generally more unit compressive strength than 8 in. dia. x 16 in. poured test cylinders.
5. Within limits, an increase in the compressive strength follows an increase in the percentage of fine material.
6. Permeability tests made on holes drilled in the concrete of the dam through a days work plane showed no seepage except where honeycombing was found.
7. Setting temperatures were found to be low enough (a rise of 29° F.) to cause no particular concern.
8. Shrinkage cracks were rare. They were radial, usually on the upstream face and then only when the concrete was poured in hot weather.
9. The included air in a dry concrete is mainly in the form of large air voids and it is practically impossible to remove these voids.
10. The air voids are not interconnected to any extent.
11. An increase in the water-cement ratio improves the looks of the concrete from the point of view of air voids but decreases its weight and strength.
12. The more rapidly the concrete is placed the better the results.

Relation between strengths in tension and in compression of mortars and concretes

R. Feret, *La Revue des Materiaux de Construction et de Travaux Publics*, No. 318-326, Mar.-Nov. 1936. Reviewed by W. H. Herschel

After 50 years as chief of the laboratory of Ponts et Chaussées, Feret has published the series of papers which should be of much interest to American investigators, based on his own long experience and the tests of others. Most of the results are shown graphically. Some idea of the vast amount of data presented is indicated by the fact that of the 110 illustrations 22 are diagrams showing the relation of the compressive strength to the concentration of the cement paste, an equal number

show the relation of the tensile or the bending strength to the concentration, and 57 diagrams show the relation between strength in tension and in compression. It is possible to present only some of the conclusions reached from such numerous and often contradictory results.

If the resistance to direct pull T, is plotted against the square root of R (where R is the resistance to compression) a straight graph is usually obtained intersecting the axis of T at a negative value, so that the equation of the line is $T = a\sqrt{R} + T_o$. A similar equation is obtained for F, the resistance to bending, but when the usual test methods are employed, the values found for F are almost double those for T. It may seem paradoxical that there is a definite value for R when $T = O$, but this only shows that the theoretical conditions upon which calculations are based do not obtain under the usual test conditions.

If a cement has been selected, it serves no purpose to impose specifications for strength both in compression and in tension, as one can not hope to obtain a product which for example, for the same compressive strength will have a markedly superior tensile strength.

The composition of the mix has an effect upon the relation between T and R. If c, e and v are the absolute volumes of the cement, water and air voids respectively, then $\gamma = \frac{c}{c + e + v}$ is merely the absolute volume of the cement per unit volume of the cement paste, including air voids, and may be called the concentration of the cement paste. Various authors have used the ratio c/e or its reciprocal, which, while it does not necessitate determining the volume of the air voids, has far less significance than γ.

If T, F or the square root of R is plotted against γ, a straight line is obtained if the richness of the mortar or concrete is not too widely different from current practice. The parameters of these lines vary with many conditions which are to be considered. For example, for two series of mortars of varying richness, made with two sands, and tested by bending at 14 days, parallel graphs were obtained having the equations $F = 158\,\gamma + 5.0$ and $F = 158\,\gamma + 1.9$, showing a straight line relation between richness and resistance to bending, and that for equal values of γ there was a difference of 6.9 kg/cm² due to the difference in sands.

It has been found in many tests that changes in the grading of the aggregates, or the addition of powdered admixtures, even in large amounts, does not change the relation between γ and strength provided the largest or the smallest particles do not predominate, but the work of Abrams shows there are limitations to this rule. We find from his tests, $F = 3.42\sqrt{R} - 10.9$, both for 7 days and for 1 year, but this relation fails if, in tests at 7 days, the *F. M.* exceeds 5.65, or exceeds 5.25 in tests at 1 year. Similarly from tests of Gonnerman and Shuman, $F = 2.9\sqrt{R} - 8.5$ is valid only up to a *F. M.* of 5.75.

In some tests of Bolomey, four aggregates were graded according to the formula $p = 10 + 90\sqrt{d/D}$, p being the per cent of particles less than d in diameter, and D being the diameter of the maximum sized particles in any one mix. D had values of 2, 4, 8 and 20 mm. respectively for the four aggregates. For these concretes, $\sqrt{R} = 52.2\,\gamma - 1.8$, for all values of D, but on account of the effect of D upon the resistance to bending, the relation between F and $\sqrt{R}$ changed from $F = 4.35\sqrt{R} - 13.0$ to $F = 4.35\sqrt{R} - 21.2$ as D was increased from 2 to 20 mm. From these and other tests it may be concluded that as the maximum size of particle is increased, F tends to decrease with respect to both γ and R, although the law of linear relation between

$\sqrt{R}$ and γ appears to be influenced but little.

Apart from puzzolanic action, the differences between aggregates can only be explained by the shape of particles, and the state of their surfaces, more or less fissured and more or less porous, when comparing aggregates of the same grading. Tests to show the effect of differences in form and nature of aggregates have not been sufficient to explain the marked differences observed among natural sands.

Within certain limits, the straight line relation between $\sqrt{R}$ or F and γ, and between F and $\sqrt{R}$ is not affected by changes in the amount of mixing water. However in tests of Richart and Bauer when $\sqrt{R}$ or T is plotted against γ, the dry mortars fall on one graph and the wet ones on another. Similarly in tests of Gonnerman and Shuman, with an increase in e/c (by weight) of 0.35 to 0.40, F increased along the graph $F = 2.91\ \sqrt{R} - 18.7$, but between values of e/c from 0.45 to 0.95, there was a decrease in F along the parallel graph $F = 2.91\ \sqrt{R} - 7.2$. These tests however differ from those of Richart and Bauer in that the relation between F and R varies with the amount of mixing water. In view of such reported effects of varying the amount of mixing water, no general conclusion is possible.

In many series of tests, when T or F is plotted against $\sqrt{R}$ for a given mortar or concrete stored in the same surroundings for different lengths of time, a straight line relation is obtained. Tests on two mortars cured in sea water and broken at different periods up to 15 years, are represented fairly well by the equations $F = 7\ \sqrt{R} - 14.0$ for a 1:5 mortar of fine sand, and $F = 7\ \sqrt{R} - 22.0$ for a 1:4 mortar of coarse sand. Other samples of the latter mortar were cured for the same periods in four other media, with the following results:

Means of Curing	Equation for F
Exposure on roof	$F = 6.18\ \sqrt{R} - 18.7$
Almost saturated air	$F = 5.65\ \sqrt{R} - 16.6$
Relatively dry air, variable temp.	$F = 4.75\ \sqrt{R} - 13.0$
Fresh water, constant temperature	$F = 4.75\ \sqrt{R} - 13.0$

It may be noted that for equal values of R, the test pieces of the same mortar, stored in sea water, gave higher values of F than obtained by any method of curing given in the above table.

In other tests, especially in the case of rich mortars, after a certain time, which may be as long as two years, there is a decrease in T or F, although R continues to increase.

Tests in which concrete was stored in various media led to the conclusion that observed differences in strength did not depend upon the effect of the media on the hardening of the cement. This conclusion was confirmed by tests on three kinds of stone. It was found that the strength decreased as the pores were more obstructed by a liquid. Without doubt this is due to the fact that the liquid, more than the air, facilitates the transmission of stresses across the mass.

To summarize what has been said, if concretes or mortars in a series differ only in a small number of factors, $\sqrt{R}$ usually varies in a straight line relation with the concentration of the cement paste, γ. The tensile strength, T or the transverse strength F, are in general linear functions of γ, but these tests are more greatly influenced by accidental variations in the concrete or in the conditions of test.

There are various causes both in molding and in curing which tend to make con-

crete lacking in homogeneity. Test pieces long stored in sea water are weakened on the surface, which perhaps is covered with fine fissures, starting rupture in a tension or bending test. In other cases crusts may form on the surface. Since any influence which makes the test pieces harder on the surface than at the center will increase F and increase the ratio of F to R, without having any marked effect on R, it is the compressive strength which should be considered as the true criterion of the hardness. If for particular purposes an exact knowledge of F is needed, it should be remembered that this is influenced considerably by the effect of the surrounding conditions on the surface of the test piece, as well as the ratio of this surface layer to the total volume, Hence reliance on values of T and F as obtained from small sized laboratory specimens should be avoided, and test pieces should be used little different in dimensions from those of the contemplated work, and cured under the same conditions.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MAY-JUNE 1937

Notes on Inspection of Structures in Europe*

BY A. J. BOASE†
MEMBER AMERICAN CONCRETE INSTITUTE

ONE OF the most important developments in reinforced concrete taking place in Europe is the work being carried on by E. Freyssinet in France. Mr. Freyssinet feels that he has produced an entirely new construction material, which he calls "treated concrete." He states that he has been working on the fundamental ideas involved since the beginning of his professional career, and in 1928 resigned his position as Technical Director of the Society of Limousin to devote his entire time to furthering this remarkable work. Any one interested in full details should read "Une Revolution dans les Techniques du Beton," par E. Freyssinet, Libriarie de l' Enseignement Technique, Paris, 1936.

The largest field application to date was recently completed at the Marine Station of Havre Harbor. This station (Fig. 1) is 1950 ft. long, 150 ft. wide, and consists for the most part of two floors, designed for a load of 500 lb. per sq. ft., and a flat roof. The lower floor (on ground) may carry as much as 1100 lb. per sq. ft.

The structure was originally built on cast in place piles which were seated on a thin layer of gravel above a soft stratum. When the structural work was completed it was observed that the building and the supporting gravel were gradually settling, in some places $\frac{7}{16}$ in. per month, and up to as much as $\frac{7}{8}$ in.

Freyssinet was called in to propose methods to stop this differential and serious subsidence. He proposed seating piles on the hard pan, which would require them to be about 100 ft. long. It was further decided that driving was out of the question since this would probably

*Presented by the author at the 33rd Annual Convention, American Concrete Institute, New York, Feb. 23-26, 1937.

†Manager Structural Bureau, Portland Cement Association, Chicago.

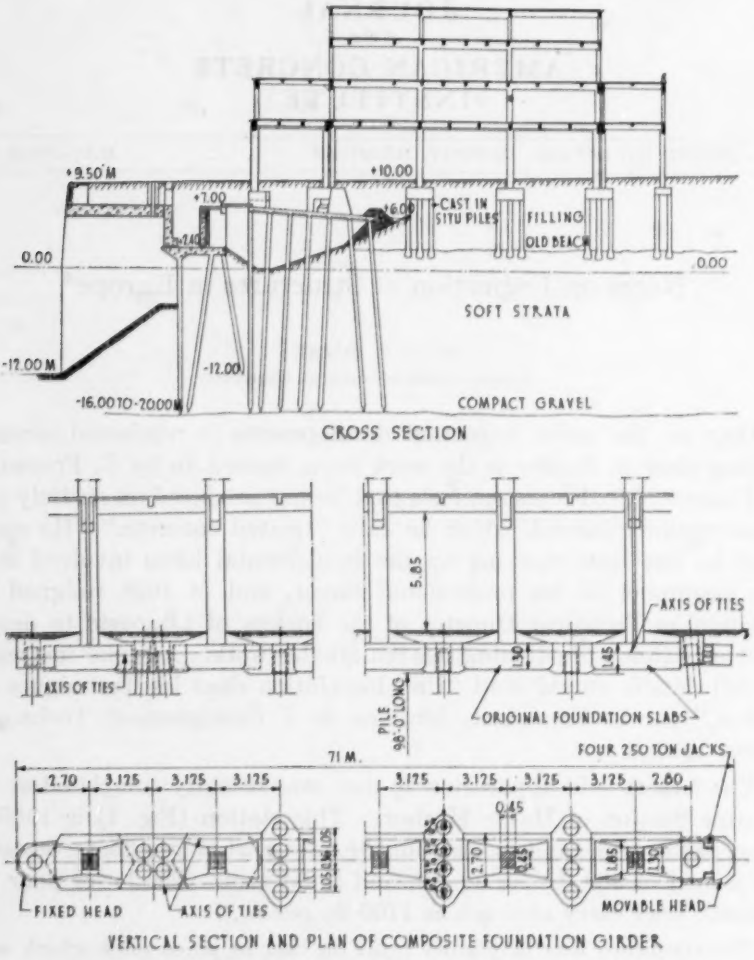


FIG. 1 (ABOVE) AND FIG. 2 (BELOW)—HAVRE MARINE STATION

disturb the mud and possibly destroy the quay. Had ordinary concrete piles been used, 2000, carrying 75 tons each, would have been required, each about 100 ft. long, and it was felt that the cost of equipment and piles would have been excessive. He therefore proposed to utilize "treated concrete," permitting the load to be carried on approximately 475 piles, each carrying about 300 tons.

Essentially, "treated concrete" consists of a well proportioned mix, vibration, pressure, and pre-stressing of the steel. The following, describing the Havre job, illustrates the process as used in the field.

The columns of the building rest on reinforced concrete slabs supported by piles, the dimensions of the largest slab being 14 ft. 6 in. by 11 ft. 3 in. by 4 ft. 6 in. (Fig. 2). The first effort was to incorporate these existing slabs into huge continuous beams capable of transferring to the future new piles the load from the original foundations and to supply the necessary resistance for sinking the new piles by hydraulic jacks. This was accomplished by forming, between these slabs, blocks of concrete reinforced only against secondary stresses. Later, in the whole of the continuous slab thus formed, a general compression was induced by means of tie rods stressed up to some 80,000 p.s.i. These tie rods were made of steel with an elastic limit of about 113,000 p.s.i., accomplished by cold working ordinary structural grade steel. The ends of these ties were anchored in concrete blocks, one block being fixed against one of the ends of the aforementioned continuous slab, while the other was supported from the other end of the continuous slab by hydraulic jacks able to give the desired tension.

When the required tension had been given, the movable concrete block was firmly secured in position and the jacks removed. The magnitude of these tensions generally reached about 1,000 tons. In

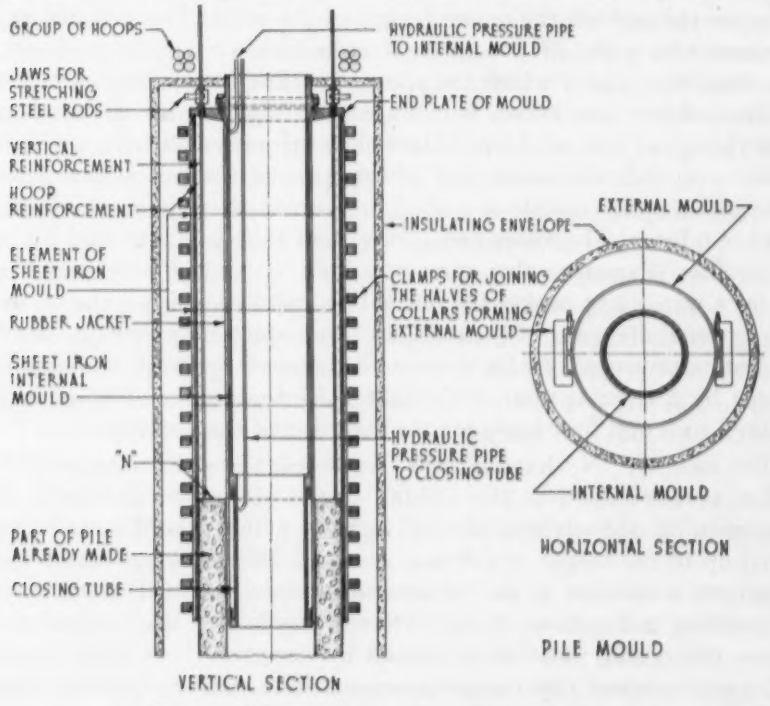


FIG. 3—MAKING A FREYSSINET PILE

this manner, huge girders able to resist bending and torsion moments as well as shearing stresses were created.

In the new concrete blocks, circular shafts provided with horizontal grooves were left, and through each of these shafts piles manufactured above ground by continuous process were sunk into the ground through the action of hydraulic jacks temporarily fixed to the girder by means of draw bolts screwed into nuts formed in the girder at the time of the concreting.

The piles are hollow cylinders of about 24-in. external diameter, 15-in. internal diameter, with a useful area of 27 1sq. in. They are reinforced longitudinally by eight wires of hard grade steel 8 mm. in diameter, and transversely by hoops of steel of the same quality 6 mm. in diameter. The total weight of the reinforcement is under 7 lb. per running foot of pile. In spite of this light reinforcement these piles are carrying 300 tons, and at times withstand a bending moment of 160 foot-tons.

As to the process of manufacture: Assume the pile molded and driven to the point "N" (Fig. 3). The upper part of the rods making up the longitudinal reinforcement comes to the job coiled and is continuous throughout the entire length of the pile. The external mold consists of a series of five to eight cylindrical collars, each about 16 in. deep, the ends of which are planed for close fitting, and each collar is divided into two halves which are fixed together by screw clamps. For the upper part of the pile the collars are provided with horizontal grooves so that the outer face of this part of the pile is corrugated. The inside mold consists of a steel tube surrounded by an envelope of rubber fabric. The lower end of the steel tube is lengthened by one of smaller diameter, also surrounded by a rubber envelope which forms a watertight pocket the normal diameter of which is the same as the external diameter of the steel tube but which may be expanded by hydraulic pressure. At its upper end the space between the molds is closed by a circular plate with eight holes to accommodate the longitudinal rods and four holes for the insertion of the concrete.

The element "N" having been completed, the screw clamps of the collar were loosened as the sinking of the pile proceeded (with the exception of the clamps on the highest collar), the internal mold lifted up to the height of the next element, and the longitudinal reinforcement connected to the transverse reinforcement. Having placed in position the collars of the external mold, the longitudinal rods above the closing plate were gripped by means of jaws held in pairs by supports which, by means of screws, were used to put the longitudinal bars into tension and to hold the reinforcements in position.

By hydraulic pressure the lower pocket of the internal mold was then expanded so as to avoid any escape of the concrete between this pocket and the internal face of the completed piles so as to close the joint between the steel tube and the rubber sheet.

The mold, thus being secured in place, was filled with a 1:1.33:2.66 concrete of sulphate-resistant portland cement, and the concrete was then vibrated by means of external vibrators attached to the external collars. About 40 per cent of the water used in mixing escaped through the joints between the collars, and part rose to the upper surface of the concrete. The water which gathered at the top section was allowed to escape through pipes in the annular plate to which the screw clamps were attached.

After all excess water had been drained off, hydraulic pressure of about 285 p.s.i. was applied between the steel tube and the rubber sheeting. Freyssinet states this pressure was transmitted by the concrete to the closing plate as it would have been through a liquid. The plate rose, thereby stretching the longitudinal rods, inducing in them a stress not far distant from their elastic limit. This pressure was maintained for about 20 minutes. The mold was then surrounded with an insulating envelope into which steam at atmospheric pressure was injected. The temperature of the concrete rapidly rose to over 212° F., and after some three hours the concrete had acquired a strength similar to that of excellent ordinary concrete several months old—this in spite of the fact that the cement used was one designed for sulphate resistance and normally gives a low crushing strength.

Immediately after the steam curing, the pile was sunk by means of hydraulic jacks. Theoretically, it necessitated a compression of about 320 tons. To this must be added the pre-stressing at the rate of 285 p.s.i., thus giving 2,850 p.s.i. Laboratory tests showed that the concrete reached a compressive strength of about 4,250 p.s.i. after three hours of heating by steam at 212° F. Actually, the piles during sinking were subject to bending moments of considerable magnitude which may have reached 160 foot-tons, and may have brought the stresses up to somewhere around 7,000 p.s.i.

Mr. Freyssinet states that piles 100 ft. long were fabricated and sunk in four working days, and that the job progressed satisfactorily and the cost was reasonable.

Freyssinet has utilized "treated concrete" in a number of places and is now manufacturing pipe near Paris by means of a patented machine based on the experience gained at Havre. He expects to make three 20-ft. lengths of pipe per eight-hour shift per machine. I happened to see one piece of pipe 30 in. in diameter, 20 ft. long, with

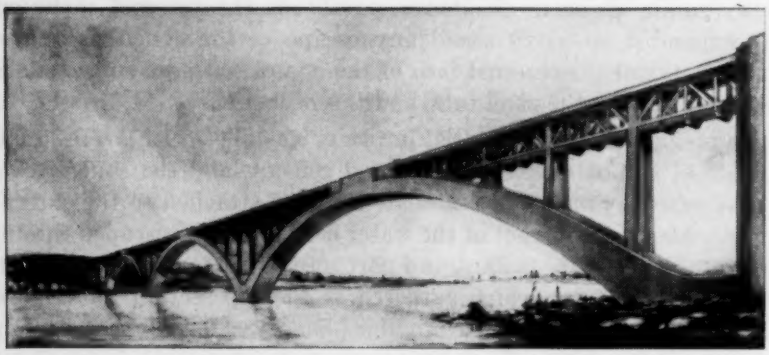


FIG. 4—THE BRIDGE NEAR BREST

a 2-in. shell, being handled around the plant at the age of three hours. He has done a great deal of work with the manufacture of poles for transmission lines, has precast girders, has built at least one testing machine, which was described in *L'Enterprise Francaise**, and feels that the method has unlimited possibilities in the structural field.

The question of pre-stressed steel is now a live one in Europe. Roof trusses using pre-stressed steel are being constructed in Germany, and many engineers abroad are seriously studying the problem in connection with its application to bridges.

LONG SPAN HOLLOW-MEMBERED CONCRETE BRIDGES

In France, one is impressed by the large percentages of bridges of the arch type. The most noted of these is the Albert Louppe (Plougastel) Bridge near Brest (Fig. 4).** This monumental structure consists of three spans of 612 ft., center to center, supporting two decks the upper one being for a highway and the lower designed to carry a standard gauge railroad. The approaches to this deck have never been completed and probably never will carry the traffic for which it was designed. Originally it was intended to carry a scenic railroad, but the increase in popularity of the automobile has prevented this occupancy.

Other notable structures are the St. Pierre-du-Vauvray, with a span of 433 ft. across the Seine, and the La Roche-Guyon (Fig. 5) over the same river. This bridge has a span of 528 ft., rise of 75½ ft., and is the longest through arch bridge in the world.

The arch ribs are fixed, having a hollow cross section (Fig. 6), and are reinforced with about 1½ per cent of spiral steel and about ½ of

*Reviewed by P. H. Bates, *Journal* Amer. Concrete Inst., Jan.-Feb., 1936, *Proceedings* Vol. 32, p. 397.

**"The 600-Ft. Arch Bridge at Brest, France," by E. Freyssinet, *Proceedings*, Amer. Concrete Inst., Vol. 25, p. 83.

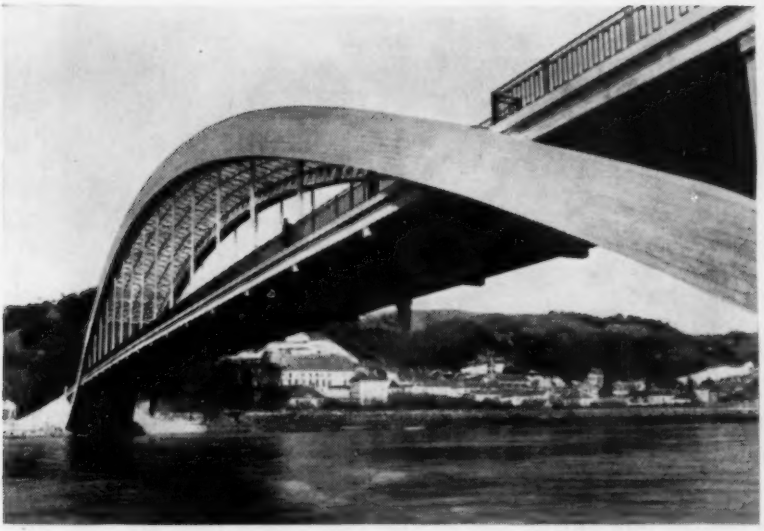


FIG. 5—ANOTHER NOTABLE BRIDGE—LAROCHE-GUYON (SEE FIG. 6)

1 per cent longitudinal steel. The longitudinal steel acts principally as spacers for the spiral reinforcing. Careful studies were made and hollow sections so designed as to give a uniform stress along the rib of about 1775 p.s.i.

All of these bridges were designed by Freyssinet, and hydraulic jacks were used for decentering. While this method of construction was invented by Freyssinet, he does not rely on it to permit long span construction. Freyssinet thinks in terms of specific weight, which he defines as the weight of a unit one meter long having a cross section large enough to carry an axial load of one metric ton. Specific cost is defined as the cost of such a unit.

The specific weight is not well defined in reinforced concrete because of the possibility of varying the essential ingredients—cement, aggregate and reinforcing steel—and the possibilities of reducing specific weight are largely dependent upon improvement in quality by properly selecting aggregates and grading them, by control of the water-cement ratio, and by use of improved methods of placing, as for instance, by vibration. By proper control of these particulars it is possible to produce concretes with strengths of 15,000 p.s.i. and with certain aggregates 20,000 p.s.i. (8 in. cubes 90 days old). These methods, he claims, do not add greatly to the cost, but are largely a matter of organization and therefore the specific weight, as well as the specific

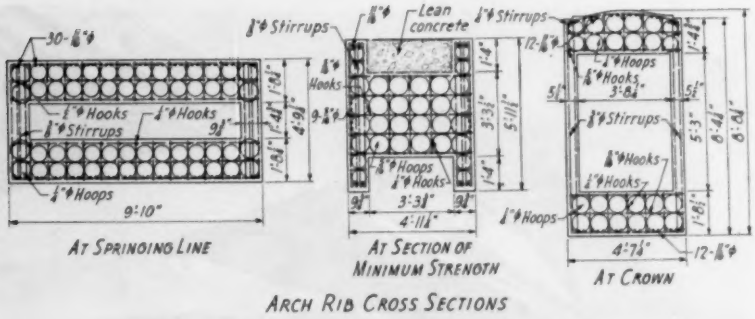


FIG. 6—DETAILS OF BRIDGE SHOWN IN FIG. 5

cost, can be reduced to as much as 20 per cent of values accepted as standard today.

Freyssinet believes that spiral compression members are never brittle no matter how brittle the several ingredients that are combined in their make-up, that is, members reinforced against lateral expansion are capable of large plastic deformation. (Fig. 6 indicates the application of this idea in the design of the LaRoche-Guyon Bridge.)

Freyssinet states that it is entirely possible, using present methods of making concrete and by the use of hydraulic jacks to relieve effects of shrinkage, plastic flow and other secondary stresses, to build solid arches of thousand-foot spans. However, he feels that the additional cost of formwork required for forming of hollow members is amply justified by saving in materials and foundation costs.

CONTINUITY IN BUILDINGS

Since the adoption of the 1936 American Concrete Institute Building Code (501-36T) and the issuance of the Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, much has been said regarding continuity in building frames. An example (Fig. 7) of this type of design and thinking is in the eight-story building, the Highgate Apartment in London, designed by the French architect Lubetkin. Here the designer has taken full advantage of concrete, both as a structural and architectural medium. The exterior walls are of architectural concrete 4 in. thick and insulated with 1 in. of asphalt impregnated cork, to which the plaster is applied. The 4-in. reinforced walls are constructed without wall columns and are built monolithically with a 4¾-in. floor slab. The longitudinal partitions in places are 8 in. thick, of reinforced concrete, and act as interior columns with a connecting beam 8 in. wide. Mr. Lubetkin's

FIG. 7—HIGHGATE APARTMENT HOUSE, LONDON, DESIGNED BY FRENCH ARCHITECT, LUBETKIN

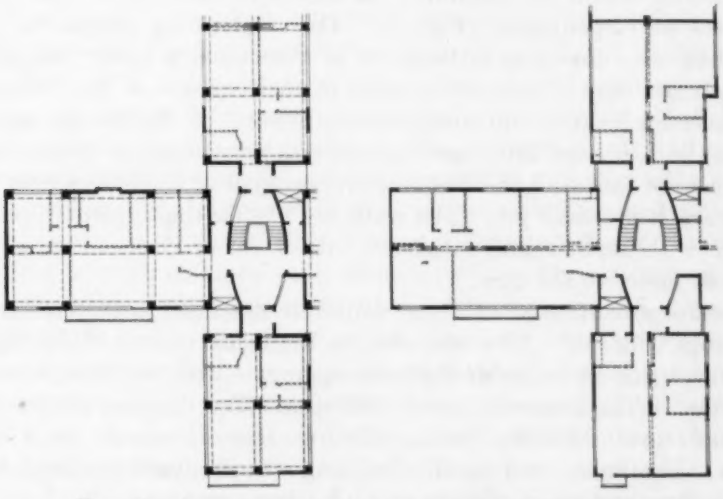


FIG. 8—A WING OF HIGHGATE APARTMENT HOUSE

drawings of one wing of this structure (Fig. 8) show the type of framing used by him on this job and also a typical American layout with wall columns for the same building. Mr. Lubetkin states that the walls, as constructed, have an insulating value equivalent to 38 in. of brick and that they have proved satisfactory in every way. The only treatment given the exterior was a coat of white cement paint.

FIG. 9—HOTEL ASTORIA, COPENHAGEN

Another lesson in continuity in concrete buildings is the Hotel Astoria in Copenhagen (Fig. 9). The engineering design for this building was done by E. Schmidt of Copenhagen using Osterfeld's deflection method. One often hears of the fondness of the European engineer for lengthy and complete calculations. I had the pleasure of going through some 240 closely spaced typewritten pages of calculation which were filed with the Building Department of Copenhagen on this comparatively small job. The walls are 4½ in. thick, painted on the inside with asphalt and insulated with 1 in. of cork. Plaster was applied direct to the cork.

Reinforcement used in these walls gives special consideration to openings (Fig. 10). The mix used in the concrete was 1:3½:3½ by volume, using ½-in. crushed granite aggregate with as "little water as possible." The concrete tested 5600 p.s.i. The designer stated that he had some difficulty setting window frames (metal) in 4½-in. walls. The finish used on this building was produced by hand bush hammering and was as fine as any job I have ever seen.

The reactions of various designers to considerations of continuity in building frames are interesting. All of them stated that they experienced no great difficulty in placing additional negative steel required and one engineer, in charge of design for one of the largest contractors, who had made many studies on the subject, stated that a saving in the frame of a building of approximately 10 per cent could be effected by careful analysis over arbitrary coefficient methods.

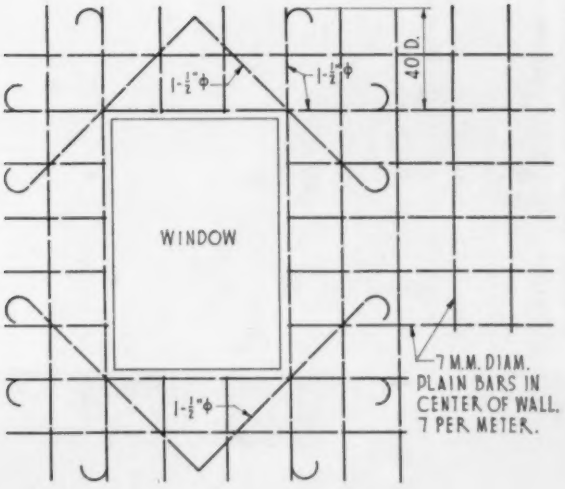


FIG. 10—REINFORCEMENT OF OPENINGS, ASTORIA HOTEL

ARCHITECTURAL CONCRETE

In all countries visited (England, France, Germany, Switzerland and Denmark), architectural concrete is making rapid strides. The architects with whom I talked were enthusiastic regarding its possibilities and continued growth. There, as here, due to its plastic possibilities, architects are constantly striving for new effects and greater economy. Surface textures or finishes continue to intrigue the profession, and one sees imitations of brick, stone and other materials, but the majority of structures examined were definitely designed with the thought of using concrete as an architectural medium in its own right.

An apartment building, Aspley House, Marlborough Road, London, (Fig. 11) built by Harold Dyke used pre-cast concrete slabs 3 in. thick, for outside forms and 5 in. of placed concrete was bonded with these slabs to make up the exterior walls. These 3-in. slabs were cast face down on wooden forms to which was attached rubber tubing to give a brick pattern, the first half inch being a red mortar made with mineral oxide and some type of waterproofing, then another half inch of porous red mortar was placed, and on top of this two inches of ordinary concrete in which ties were bedded for bonding to the structural concrete. The slabs were made about 2 ft. by 3 ft. in area, and after erection, white cement mortar was used for pointing to give the jointed appearance of brick. This job proved so economical that the contractor, Mr. Dyke, was awarded a large government housing development using the same system of construction.

FIG. 11 (LEFT) — ASPLEY HOUSE, MARLBOROUGH ROAD, LONDON, BUILT BY HAROLD DYKE

FIG. 12 (RIGHT) — A. G. PERRET'S HOME AND OFFICE IN PARIS

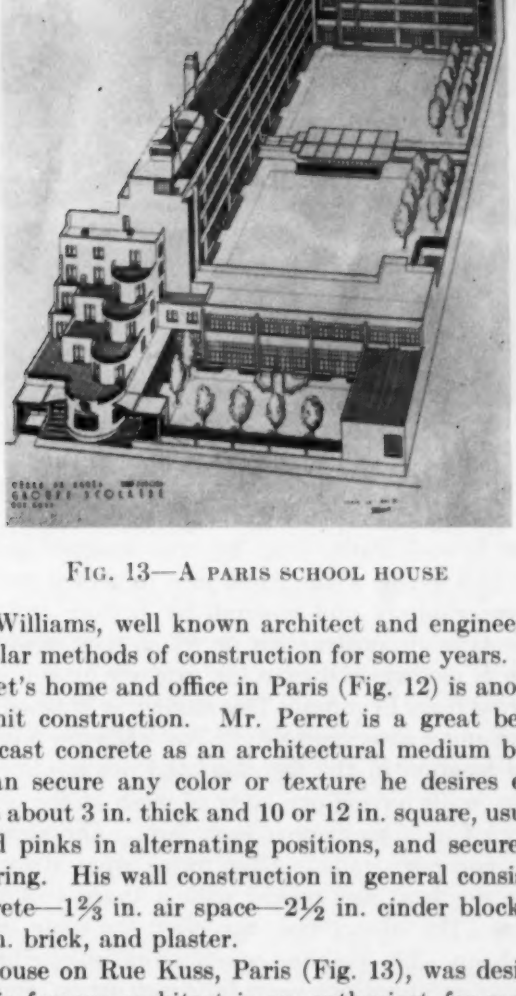


FIG. 13—A PARIS SCHOOL HOUSE

Sir Owen Williams, well known architect and engineer of London, has used similar methods of construction for some years.

A. G. Perret's home and office in Paris (Fig. 12) is another example of precast unit construction. Mr. Perret is a great believer in the future of precast concrete as an architectural medium because, as he stated, he can secure any color or texture he desires economically. He uses units about 3 in. thick and 10 or 12 in. square, usually employing buffs and pinks in alternating positions, and secures texture by bush hammering. His wall construction in general consists of 3 in. of precast concrete—1⅔ in. air space—2½ in. cinder block—1⅔ in. air space—4½ in. brick, and plaster.

A school house on Rue Kuss, Paris (Fig. 13), was designed by Mr. Expert. This famous architect is an enthusiast for monolithic construction and has done some splendid jobs. At present he has under construction at Metz a church in parabolic form of exposed concrete that is attracting a great amount of attention because of its unusual functional design. Mr. Expert uses wall construction generally of 4½ in. bush hammered monolithic concrete—4½ in. of brick back-up —1⅔ in. air space—3¼ in. gypsum block, and plaster.

FIG. 14—CAMILLE SEE SCHOOL, PARIS

Camille See School, Paris, (Fig. 14) was designed and constructed by LeCoer. This is one of the finest jobs inspected and is the result of some 20 years' study and research by LeCoer and his father to determine the type of construction on which they would concentrate. This concrete is a warm red in color, created by use of a dark red marble imported from Belgium. This stone was crushed and specially graded for fine and coarse aggregate. The walls are 5 or 12 in. thick, depending on location in the building, and are reinforced with wire mesh, the 5-in. walls having one layer of mesh at the center, and the 12-in. walls a layer of mesh near each face. The plaster is applied direct to the concrete walls. After forms were stripped, the wall surface above the base was picked with a sharp tool by hand to a depth of $\frac{3}{8}$-in. The base course was given the same treatment, but to a depth of $\frac{3}{4}$-in. The building covers an entire city block, and cost 24 million francs, required two years for construction, and was finished in 1934.

A newspaper building in Stuttgart, Germany, (Fig. 15) an architectural concrete job of 17 stories, was the tallest building seen on the trip. The walls are 4 in. thick, of gray cement and crushed stone, with a bush hammered texture. Wall insulation is 1 in. of pressed peat, to which the plaster was applied.

FIG. 15—A NEWSPAPER BUILDING IN STUTTGART

INDUSTRIAL AND COMMERCIAL BUILDINGS

From an engineering point of view, some of the most interesting buildings fall in the industrial and commercial class. Here the French have displayed greater daring and ingenuity than any other people visited. At the present time the Germans are making rapid strides with "shell" construction and are building some most unusual structures.

It is difficult to determine why these countries have been leaders in this field. The usual reason given is that labor there is relatively cheap and materials are costly. I found that common labor was getting 30 to 35 cents per hour, and the most skilled mechanics were being paid about 15 to 25 per cent more. Materials, generally speaking, cost about the same there as here, but I doubt that saving of materials is the final answer. An examination of the walls shown in the buildings pictured would not indicate this to be true. It seemed to me, when talking to engineers and architects, that fundamentally there was a real difference in their philosophy as contrasted with the American viewpoint. To them concrete is an old and accepted material. It is neither regarded as a substitute nor as an imitation, and long use

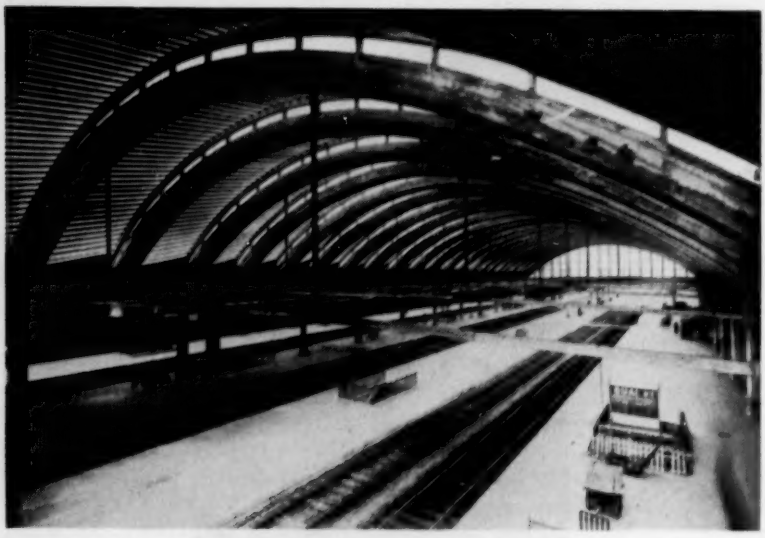


FIG. 16—THE STATION AT REIMS

FIG. 17—EXTERIOR DETAIL OF ARCH RIBS

and successful experience have proved it to be an economical medium, both structurally and architecturally.

Their system of contracting may have been a real influence. I found it rather difficult to find structural engineers in private practice as we have them in this country. Usually the better men are in the employ of contractors, and some of the most interesting and unusual jobs are designed by contractors' engineers. One experience in France which seemed typical will illustrate the procedure. An architect had prepared floor layout and specifications and had drawn elevations of a given job, and issued it for bids. The specifications stated that a reinforced concrete frame would be used, but no structural plans had been prepared. The columns had not even been spotted. This meant that each contractor bidding would not only prepare an estimate, but he would make his own structural design. In this he was allowed much freedom, so that it was possible for him to plan to utilize his equipment and personnel in the best manner. Under such conditions he could produce an economical, and possibly an unusual design. This does not mean that the contractor can violate good practice. The building departments seem to be well equipped with technical talent and a thorough check is probably made.

The laws, too, are rather strict. The architect and contractor are responsible for the performance of a building for ten years after its erection. Mr. Perret told of a building which had cost 17 million francs. After being put in use it was found to be faulty, and the courts assessed the contractor 5 million francs and the architect 12 million! Of course, there are insurance companies which will issue insurance to architects and contractors as protection.

The station at Reims, (Fig. 16 and 17) is a good illustration of design produced by competition. The Eastern Railroad, through its architect, required, among other conditions, (1) out of 6800 square meters, 6000 should be glass for light, (2) free passage of 335 square meters for smoke exit, (3) permanent and rapid means and easy access to glass for cleaning, and (4) use of reinforced concrete for frame. From the many schemes presented, that of Anonymous Society of Limousin Enterprises was chosen. Large arch ribs are in the form of a "U" and serve as foot paths for men cleaning the glass. Steps were formed in these U-shaped ribs where the slope was steep (Fig. 17).

An unusual conoid roof construction was employed on a shop building in the Fontenay-sous-Bois yards of the Metropolitan Railroad of Paris (Figs. 18 and 19).

There is an interesting pier construction at Hamburg, Germany with a typical Z-D shell roof. Originally this building was designed in

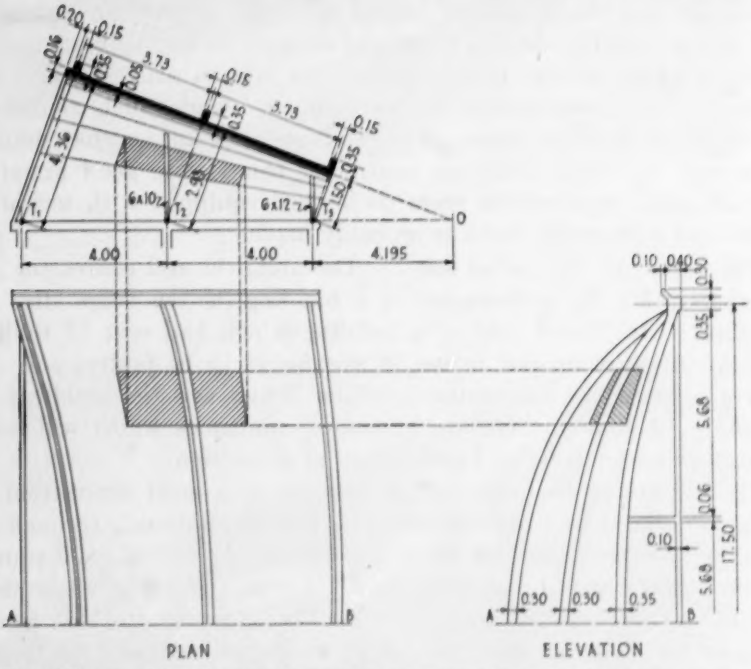


FIG. 18 AND 19—SHOP BUILDING OF THE METROPOLITAN RAILROAD

wood and in competition was changed to reinforced concrete. Plans and specifications for this job were secured and submitted to contractors in the Chicago area for bids. The only change made was in the thickness of the shell roof. As built in Germany, it is 2¼ in. thick. This was made 3 in. due to fireproof requirements. Bids submitted indicated it could be built in Chicago for 90 cents per square foot of horizontal projection. The competitive design as might be used

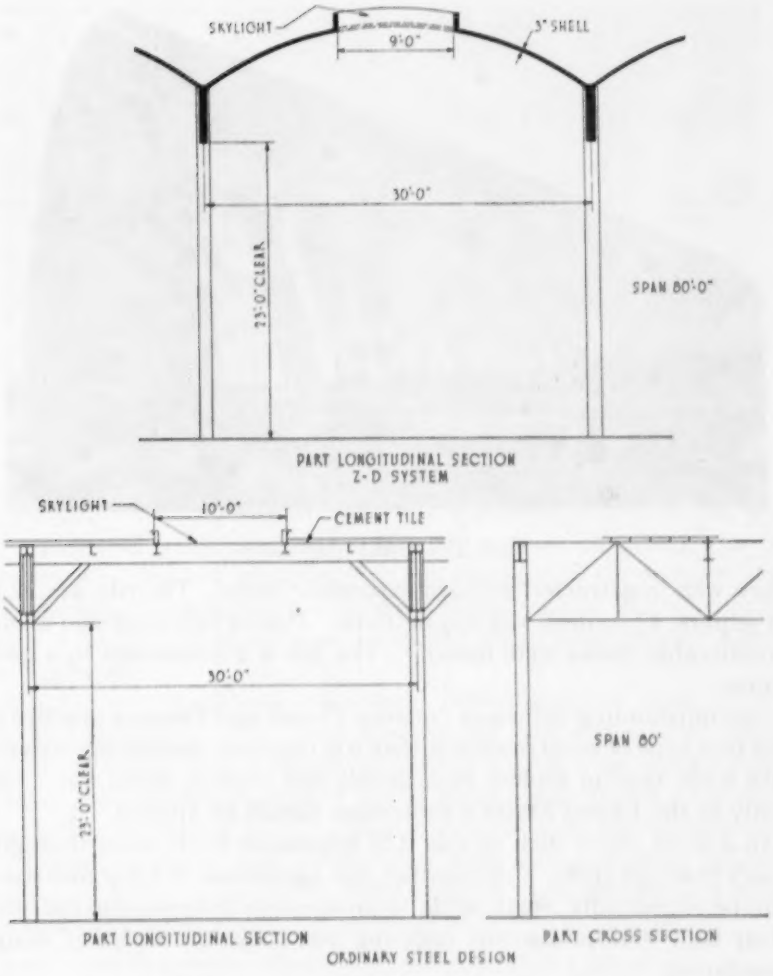


FIG. 20

in this country was likewise submitted and a comparative bid price of $1.03 per square foot of horizontal projection was secured (Fig. 20).

After the concrete work on this job was completed the work was shut down and the building stood five years without protection. It was completed only a year or so ago. A careful examination showed that the building was as near perfection as a job could be.

The most spectacular and amazing job visited was the Orly hangars in France (Fig. 21). A. G. Perret was the architect, and E. Freyssinet the engineer. There are two of these structures, paid for by German reparations, each about 1,000 ft. long, 300 ft. wide, and 300 ft. high.

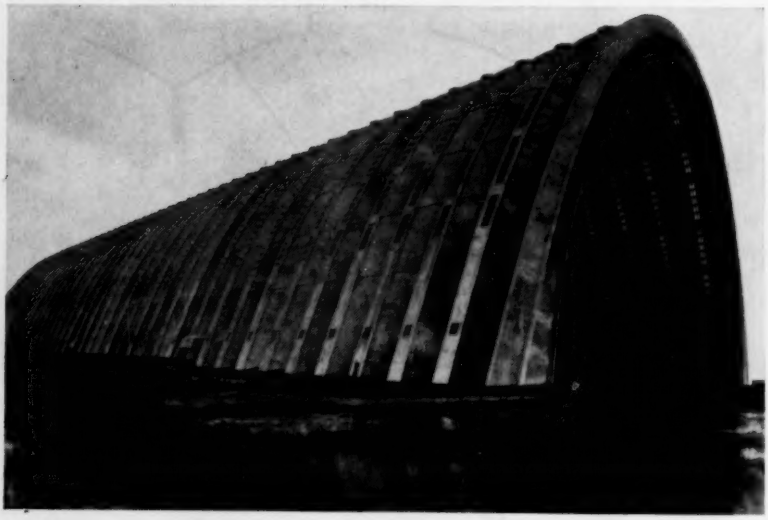


FIG. 21—ORLY HANGARS

They were constructed without expansion joints. The ribs are 22 ft. on centers, 17 ft. deep and 3½ in. thick. One or two short and almost unnoticeable cracks were found. The job is a monument to a great genius.

One outstanding difference between French and German practice on this thin type of construction is that the Germans protect the exterior with some type of roofing such as felt and asphalt, slate, etc. Certainly in the United States a protection should be applied.

In a short paper such as this it is impossible to do more than give a very few high spots. Altogether, the experience of a few months in Europe is decidedly worth while to an engineer interested in concrete. Their bold conceptions are inspiring and their knowledge of design stimulating.

Some mistakes were made, and had I to do it over, France would be the last country visited instead of the first. The transition from American ways to those of Europe might well start with England, then Denmark, Germany and Switzerland, and end with France. The French mind, though great, is not easy for an American to understand.

A great lesson can be learned from European hospitality. Many foreign engineers have called at my office, and I doubt that I have exerted myself for them as they did for us.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by August 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

(Supplement)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . SEPTEMBER-OCTOBER, 1937

Discussion of a Paper by A. J. Boase:

Notes on Inspection of Structures in Europe*

BEN MOREELL†

The paper is most interesting. I am especially pleased to note the amount of space devoted by Mr. Boase to an explanation of M. Freyssinet's "treated concrete." I am strongly of the opinion that M. Freyssinet's work has not received the attention in this country which it merits and which would certainly prove profitable for American engineers.

Mr. Boase refers those interested in details of the method to a French publication. For those who are not well equipped to handle somewhat involved theoretical discussions of basic scientific conceptions in the French language, I would like to state that I have made translations of two of M. Freyssinet's expositions of the fundamentals of "treated concrete" and descriptions of its applications to practical problems. I have a number of extra copies of these translations which I will be pleased to distribute to those who may be interested.

*JOURNAL Amer. Concrete Inst., May-June, 1937; *Proceedings* Vol. 33, p. 521.

†Commander, (C. E. C.) U. S. Navy.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MAY-JUNE 1937

Recent Developments in Foundation Design—with Special Reference to Concrete*

BY CARLTON S. PROCTOR†

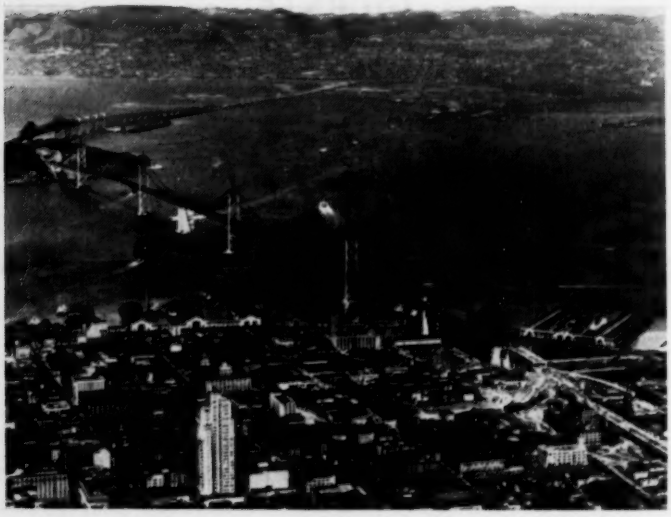
RECENT advancements in foundation engineering are of far reaching importance to the future of structural design and have developed around the freer use of concrete. A number of recent major foundation installations have employed entirely new principles, opening new fields for future design.

Probably the most outstanding example of modern bridge foundation design is found in the design of the West Bay Crossing piers for the San Francisco-Oakland Bay Bridge, Fig. 1, employing the newly invented and patented Moran Caisson. The relative dimensions of this bridge and the world's next longest, the Golden Gate Bridge, are shown by the following comparison:

Name of Bridge	Total Length ft.	Max. Height of Towers Above Water ft.	Max. Depth of Piers Below Water ft.
San Francisco—Oakland Bay Bridge....	22,720	505	235
Golden Gate Bridge.....	8,940	746	100 (Approx.)

Fig. 2 shows the actual perspective developed in the office of Moran and Proctor during the early design studies which produced the caisson design later patented and used for the West Bay Crossing. Prior to this installation, economy and feasibility had required that bridge piers be located in comparatively shallow water, so that frequently the economic considerations resulted in locating the piers near the shores, even though the resulting bridge spans were uneconomically long. To illustrate, assuming the depth of bottom of a water

*Presented at the 33rd Annual Convention, American Concrete Institute, New York, Feb. 23-26, 1937.
†Moran, Proctor & Freeman, Consulting Engineers, New York.



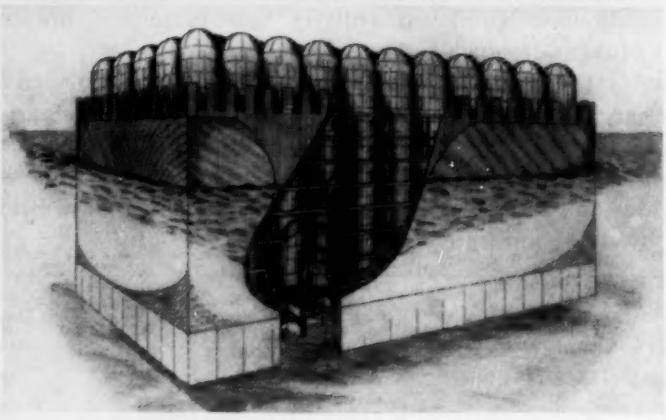


FIG. 1—SAN FRANCISCO-OAKLAND BAY BRIDGE

FIG. 2—ORIGINAL PERSPECTIVE DEVELOPED BY MORAN AND PROCTOR

crossing to increase to a maximum at the center of the crossing, the economics and feasibility of the foundation installation have previously required that pier locations be controlled by the depth of required caisson flotation rather than by the depth of caisson dredging. Not only did the cost increase rapidly as the depth of water increased, but many crossings presented water depths in excess of the permissible limit to which precedented caisson types could safely provide flotation. But the "Moran Caisson" reversed the requirements and provided a design which facilitates bridge pier installations not only in water

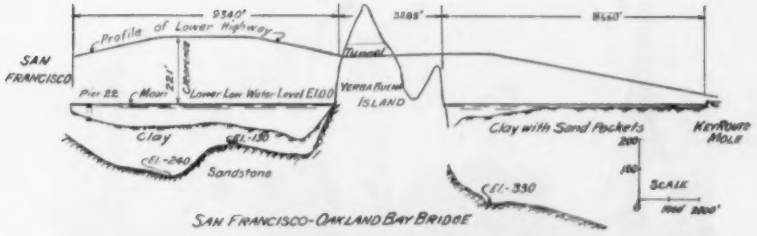


FIG. 3—PROFILE SAN FRANCISCO-OAKLAND BAY BRIDGE CROSSING

depths considerably in excess of previous limits, but where the economics of the installation are controlled by the depth of caisson dredging and not by the flotation depth. Assuming the same water crossing with maximum water depth at the center and where the bedrock, hardpan, or gravel medium for the support of piers is at a constant depth, the cost of the "Moran Caisson" decreases as the depth of water increases. Hence bridge piers may be installed in water depths heretofore uneconomical or impracticable, and the economics of such foundations will tend to produce bridge spans of maximum economy and will make bridge installations feasible in hitherto impracticable locations.

At the San Francisco-Oakland Bay Bridge the depth of open water and the depth of sinking required, in order that the bridge pier caissons reach bedrock, as shown in Fig. 3, considerably exceeded any previous bridge pier installation. The piers required a design to provide controlled flotation up to a depth of 120 ft. and to sink to a maximum depth of 240 ft.-requirements beyond the safe and economic limits of precedented methods.

The Mississippi River Bridge at New Orleans was then in process of design and contemplated piers to the then unprecedented depth of 180 ft., with maximum depth of 75 ft. of water. The deepest piers actually installed up to this date were those for the Hawksbury Bridge at New South Wales, Australia, where a maximum depth of 162 ft. had been reached, with a depth of 70 ft. of open water. The San Francisco Bay pier installation, therefore, contemplated sinking piers to a depth 50 per cent in excess of that previously accomplished, with a flotation depth of 70 per cent in excess of the Hawksbury installation. As these depths far exceeded the limits of pneumatic work, the only method available was by open dredging caissons, and all such previous installations had employed either the braced cofferdam principle or the false bottom method to provide caisson flotation. At 120 ft. hydrostatic uplift the false bottom method was obviously

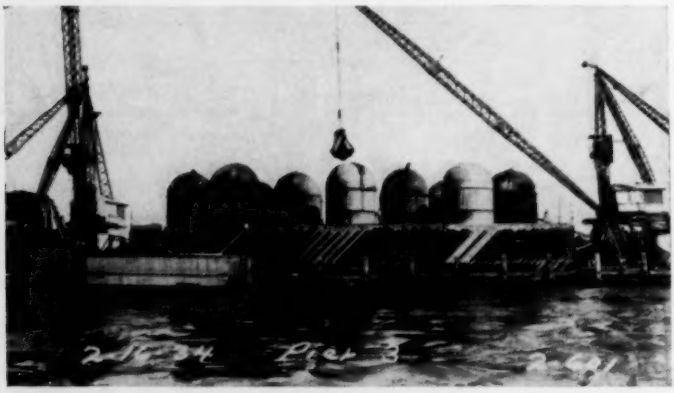


FIG. 4—SINKING PIER 3

impracticable and unsafe. The braced cofferdam principle of flotation requires an open cofferdam from one half to three fifths the depth of flotation and such an installation here would have required such heavy bracing that the body of the pier would have been seriously reduced by it, and many elements of severe hazard during installation would have been inevitable. It became necessary, therefore, to evolve a new principle of foundation installation to meet these conditions and the Moran Caisson was developed and patented.

The Moran Caisson (Fig. 4) comprises a cellular reinforced concrete open dredging pier, where pneumatic flotation provides the equivalent of a false bottom near the lower end of each dredging well, but where such false bottoms can be moved at will upward or downward within the dredging wells and are completely removed preparatory to dredging within the caisson. This is accomplished by constructing the dredging wells as circular steel cylinders, where such cylinders act as the forms for the reinforced concrete body of the pier and where they are extended well above the top of each caisson build-up and covered by steel domes, fitted with valves and connections for the introduction of compressed air and for the later introduction of water as the compressed air is vented off. By properly proportioning the area of the air filled dredging wells to that of the caissons, required flotation is obtained, but to maintain the center of gravity of the caisson well below the metacenter and thus to assure a constant positive righting moment, low head cofferdams are used.

To provide for progressive removal of the domes and the upward extensions of the steel cylinders during each caisson build-up operation, these caissons were designed to develop a 10 per cent excess

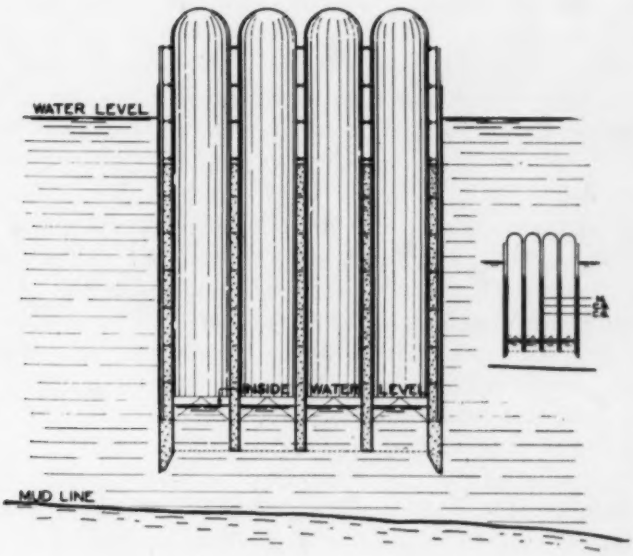


FIG. 5—MORAN CAISSON FLOATING

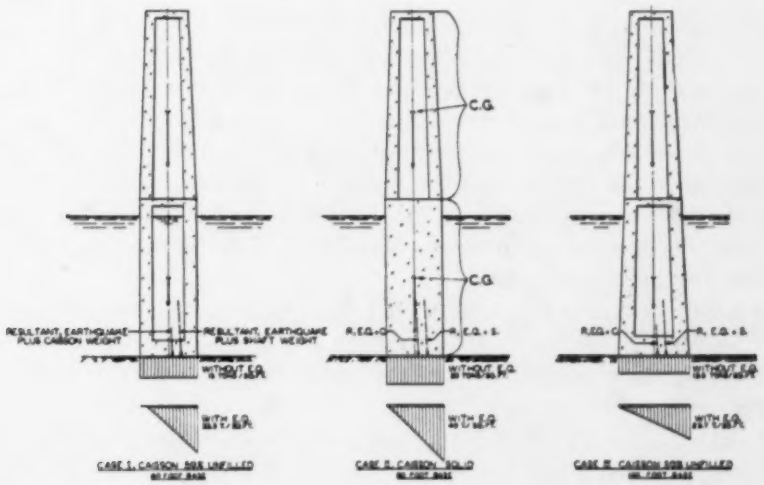


FIG. 6—RELATIVE EFFECT OF INCREASED WEIGHT AND INCREASED BASE WIDTH ON STABILITY OF BRIDGE PIER SUBJECT TO EARTHQUAKE FORCES—EARTHQUAKE FORCE ASSUMED EQUAL TO 5 PER CENT OF GRAVITY

flotation, thus permitting the removal of 10 per cent of the domes at one time. As a factor of safety and control, the caissons were designed to float with the water level within the cylinders at a minimum of 10 ft. above the cutting edge, and the design, Fig. 5, assumed that the cutting edges would penetrate into the semi-liquid mud bottom to a depth of 15 ft. before the caissons' weights were transferred to the underlying soil.

With these basic assumptions established, comparative studies were made involving various cylinder sizes and concrete wall thicknesses (Fig. 6). Each study required a thorough investigation of flotation and stability, bracing and cylinder design, stress determination for the final pier and estimates of cost. From these studies was evolved the final design which proved economical, thoroughly practicable in each stage of installation and flotation, and the fulfillment of every hope in the reduction of usual hazards and contingencies.

In addition to its positive control at all stages of flotation, this caisson has the additional outstanding advantage that, as it approaches the bottom, it may be accurately located and quickly lowered to a firm embedment in the underlying soils. (Fig. 7). Ordinary methods of caisson installation require the gradual approach of the cutting edge to the mud bottom, and therefore in swift currents there may develop a scour under the cutting edge as its distance from the soft mud bottom is reduced.

This method provides the further advantage that whenever the cutting edge is within an allowable pneumatic working depth, air locks may be installed on the domes, and workmen may enter the cylinders to remove obstructions at the cutting edge level. (Fig. 8). When the domes are in use they serve as cushions against the caisson listing, since the effective center of gravity is lowered, and the load on the cutting edge is reduced by the compressed air in the wells. When sealing the caisson, air pressure may be used to reduce the effective weight on the cutting edge.

Another notable achievement in bridge foundations was developed for the Mississippi River Bridge at New Orleans (Fig. 9). Here an unusual set of sub-surface conditions required an unusual foundation design treatment.

A maximum depth of 90 ft. of open water to a shifting semi-fluid silt bottom, required a light caisson to provide flotation, and at the same time adequate strength of construction to withstand the hydrostatic pressure on the hollow walls during flotation. The subsoil material to support these piers consists of a compacted sand which was valued at a supporting power of only 4 tons per sq. ft. in excess of existing

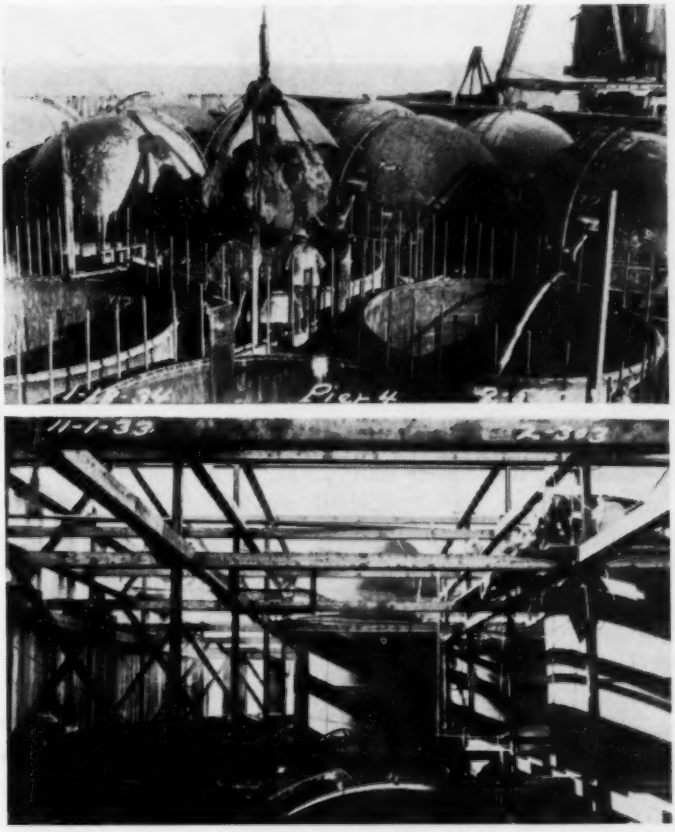


FIG. 7—INITIAL DREDGING OPERATION IN PIER 4 WITH ONLY PART OF THE CYLINDERS REMOVED

FIG. 8—CAISSON BUILT UP SHOWING EXTENSION OF CYLINDERS (10 PER CENT AT A TIME) AND EXTENSION OF BRACING SYSTEM AND EXTERIOR SKIN

soil pressure, which also required a caisson of the minimum possible weight. But opposing these requirements for maximum lightness was the requirement for sufficient weight to overcome the skin-frictional resistance encountered in sinking through alternate sand, clay, gumbo, compacted sand and more gumbo to the then unprecedented depth of 185 ft.

This problem was met by a new type of caisson design, (Fig. 10) developed and patented, which extended the dredging wells virtually out to the outer walls of the caisson, eliminating the usual thick, heavy outer wall construction, and by providing that certain portions

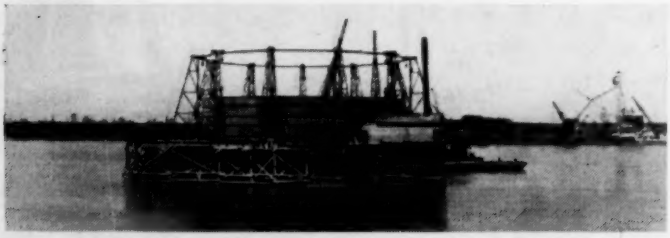


FIG. 9—MISSISSIPPI RIVER BRIDGE—REMOVING SHELL, PIER 3

of the caisson walls should be unfilled above the height required for the caisson seal. Those hollow spaces not required for permanent construction were filled with ballast during caisson sinking and the ballast removed when the caisson was landed.

These caissons, illustrated in Fig. 11, were installed by the sand island method where the steel shells confining the sand island had diameters up to 121 ft.; 40 ft. in excess of the previous record for this type of installation, established for the Suisun Bay Bridge piers.

The new science of soil mechanics played an important roll in the design of the foundations for both the San Francisco-Oakland and the New Orleans Bridges. The New Orleans Bridge would probably have been built years before it was, if it had not been for the question of the feasibility of installing stable foundations. Previous engineering studies and reports as to available sites for this bridge and the feasibility of stable foundation installation, had assumed adequate supporting materials varying from the gumbo stratum at elevation minus 150; down to a deep sand stratum at minus 250. Soil mechanics laboratory analyses and tests demonstrated conclusively that foundations on the upper gumbo stratum would be entirely unsafe, and economic studies demonstrated that foundations on the sand stratum at minus 250 would involve costs entirely prohibitive to the construction of the bridge. But these laboratory studies demonstrated the adequacy, under proper intensities and proper design treatment, of the sand stratum at minus 185 and facilitated definite knowledge of the characteristics of this stratum and the underlying materials. Thorough soil mechanics investigations and test of undisturbed samples of the soils below elevation minus 185 developed a safe unit soil bearing capacity at this level at 7 tons per sq. ft., or a maximum of 4 tons per sq. ft. in excess of the natural soil pressure and the fact was also developed that pier settlements would be less and allowable intensities greater at this level, than at the lower levels previously assumed.

FIG. 10—MISSISSIPPI RIVER BRIDGE—REMOVING FORMS FROM WELLS, PIER 2

FIG. 11—MISSISSIPPI RIVER BRIDGE—ERECTING COFFERDAM, PIER 3

Prof. William P. Kimball was in charge of the laboratory of Moran and Proctor, consulting engineers for this foundation design. In Vol. I of the *Proceedings of the International Conference of Soil Mechanics and Foundation Engineering*, Harvard University, Professor Kimball discusses the relationship between the calculated and the actual bridge pier settlements. As a result of the preliminary soil mechanics tests and analyses the truss bearing plates were designed to permit of jack-

ing up the trusses and the bridge seats were finished at elevations sufficiently high to contemplate future settlements.

The West Bay Crossing piers for the San Francisco Oakland Bay bridge employed the Moran Caisson as previously described. (Fig. 12). But the foundation problem for the East Bay Crossing, from Yerba Buena Island to Oakland, differed radically from that of the West Bay Crossing. Borings showed that rock drops off sharply immediately east of the island, to a depth which is unreachable by any prac-

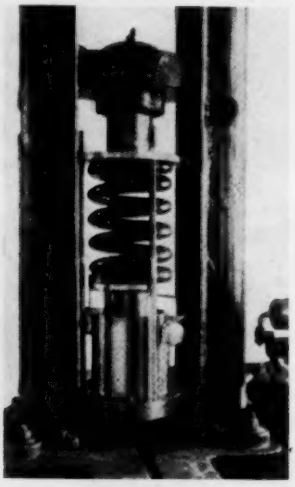
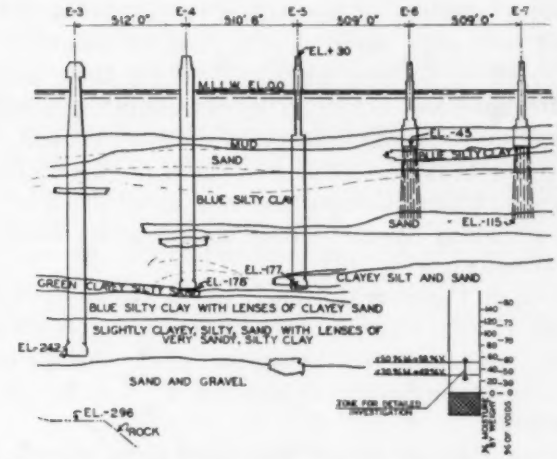


FIG. 12—IMPROVED CONSOLIDATIONS TEST APPARATUS IN POSITION IN TESTING MACHINE

ticable method of construction. For these foundations, therefore, it was necessary to determine accurately the character of the soil so that the piers might be designed to bear on an adequate subsoil stratum at safe intensities of load. In order to obtain the information required for a complete analysis of the soil, a sampling device was developed for this project, as described in the *Engineering News Record* (June 23rd, 1932).

Undisturbed samples were hermetrically sealed on the drill barge, immediately after their procurement, and shipped to the laboratory of the University of California for testing. Duplicate samples were sent to the Moran and Proctor Laboratory in New York for comparison and testing. In the laboratories, determinations were made of the structural characteristics and behavior under load of the soils to be loaded, and the character of the strata above the foundation levels



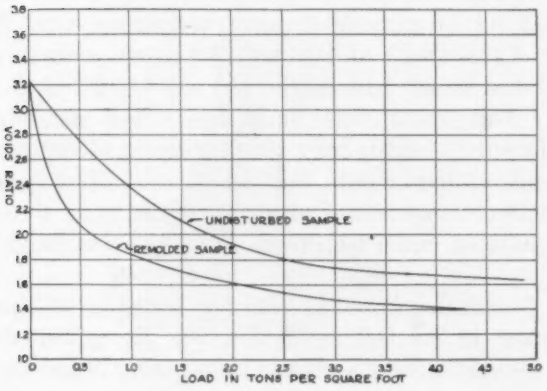


FIG. 13—FOUNDATION DATA, EAST BAY CROSSING—GEOLOGICAL SECTION

FIG. 14—LOAD—CONSOLIDATION CURVES—UNDISTURBED AND REMOULDED SAMPLES

which might affect problems of caisson sinking, excavation and construction. A complete series of routine tests were made.

In additior to the routine tests, consolidation tests on both undisturbed and remoulded samples were made to determine the design unit intensities and the probable settlements. The apparatus used in making the consolidation tests was especially designed for this work and was believed to constitute an improvement over devices previously used.

A geological cross-section, Fig. 13, under the principal East Bay piers was developed to show the subsoil materials classified on a

basis of moisture content or per cent of voids. Many soil laboratory analyses and tests were required to accurately determine the depth, bearing area, allowable load intensity, etc. for these piers, and this work was simplified and expedited by concentrating detailed investigations on soils in the lower brackets of moisture content.

Soil mechanics, that relatively new tool of the foundation engineer, is playing an increasingly important role in all types of foundation designs and is making possible certain designs which would have been unsolvable without it. (Fig. 14).

The creation of Flushing Meadow Park at the site of the Flushing River swamps with the support of structures for the New York World's Fair, and the foundations for the support of the world's tallest structure, the Palace of the Soviets at Moscow, U. S. S. R., are two excellent cases in point.

The Flushing Meadow area of over 1000 acres consisted originally of a semi-fluid silt of varying depths overlying a sand floor. The northern part of this area had been used for many years as a dump for ashes and rubbish which had caused the silt deposit to flow in large mud waves. Difficulties with foundations for structures in the Meadows had resulted largely from the flow of the silt when surcharged unevenly by fill, embankments, buttresses, etc. Mud wave action in the silt caused its remoulding with the result that the remoulded material consolidated under load from 2 to 4 times as much as did the undisturbed.

The major foundation problem was not so much the provision of adequate support of vertical loads, but the prevention and control of lateral soil movements. Careful soil mechanics laboratory analyses

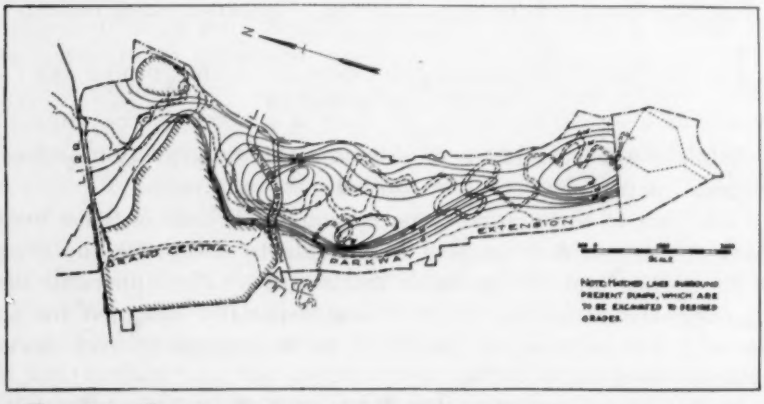


FIG. 15—CONTOUR PLAN OF ESTIMATED SETTLEMENTS, GRAND CENTRAL PARKWAY EXTENSION

developed the shearing strength of the silts and of the vegetable humus mattress overlying the undisturbed areas, and the extents to which consolidations would take place under filled areas, (see Fig. 15), so that a program could be developed for the filling and grading operations which would not cause subsoil distrubances and which was relatively economical and simple. Through the information obtained from the soil mechanics laboratory, foundation designs for the various park and World's Fair structures, such as bridges, boat basins, tide gates, diversion channels and buildings were developed which obviated the difficulties and movements experienced on many previous structures but provided stable foundations through economical and simple methods.

FIG. 16—PALACE OF THE SOVIETS—SUPERSTRUCTURE

The design adopted for the foundations for the Palace of the Soviets would have been impossible without the aid of soil mechanics. Situated just east of the Kremlin on the Moscow River, the site presented an unusual geological cross section, consisting of artificial fill—alluvial sand, shattered limestone, marl, undisturbed limestone, more marl, more undisturbed limestone, etc. successively down from the surface. Wide variations in the compressibility of the interbedded marls and sharp variations in the marl thicknesses occurred over short horizontal distances.

The foundation problem resolved itself into a question of, first, the maximum differential settlement which the structure would safely withstand; and, second, the level of bearing which would provide

such required result, i. e., the depth to which the supports must be carried.

The superstructure design (Fig. 16) is unprecedented and one which lends itself to but slight differential settlements. Most simply stated the superstructure steel skeleton consists of a single set of two leg columns placed on the perimeter of the tower base, with no interior columns. The diameter of the circle of the inner column legs is 136 meters and that of the outer column legs 148 meters. These columns set in from the vertical as they rise around the hemispherical dome of a circular auditorium 130 meters in diameter and approximately 100 meters high, by 2 successive inward breaks from the vertical (See Fig. 17). At the top of the dome the columns resume vertically, so

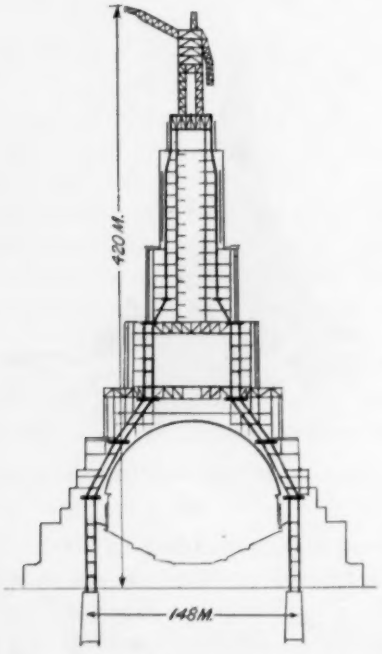


FIG. 17—STRUCTURAL FRAME, PALACE OF SOVIETS

that at this level there is a large inward horizontal component of column loads which must be resisted by an annular girder acting as a circular horizontal column. Any differential settlement at this level would cause accentric bending stresses in the annular compression girder, which unless strictly minimized are obviously inadmissible in such a design.

Applying the results of laboratory consolidation tests an undisturbed samples of the marlaceous materials lying between the second and third limestones, the amount of consolidation which would result from isolated column footings supported on these materials was calculated, together with the effect on such consolidations by variations in the intensity of loading on the upper surface of the second limestone.

Vertical components of soil stresses were calculated by the use of Boussinesq's formulae, at depths below the top of second limestone of 5, 10, 15 and 20 meters respectively.

By use of the consolidation curves for undisturbed samples, settlements under assumed isolated piers were calculated to vary from a minimum of 1.2 to a maximum of 9.3 cm., under 10 kg./cm.^{2} intensity; from 1.7 to 10.8 cm., under 20 kg./cm.^{2}; and from 1.9 to 11.2 cm., under 30 kg./cm.^{2}; with a maximum differential settlement between immediately adjacent columns of $3\frac{1}{2}$ cm. under a 20 kg./cm.^{2} intensity.

Utilizing these determinations, a plan was developed showing contours of settlement which would result from isolated column footings bearing at 20 kg./cm.^{2}, such plan being merely an interpolation of settlements arrived at by assuming that at each boring location, intensities on the various subsoil strata would be those existing under isolated tower columns if the entire ring of loaded columns were moved so that one column was situated at the location of the particular boring. This drawing also gave a circumferentially developed profile of settlements plotted from the settlement contours, and a profile of vertical consolidations based on an intensity of 20 kg./c. where the tendency to settlement was least, decreasing to an intensity of 10 kg./cm.^{2} where the tendency to settlement was greatest. Superimposed on this last profile was a sine curve of anticipated consolidation under a continuous annular reinforced concrete girder carrying the column towers, such sine curve being based on a slight anticipated tilt of the structure across a major axis between minimum and maximum marl consolidations, and such sine curve indicating the maximum consolidations permissible without departing from the required plane through the column bases and thus maintaining the annular compression girder at the top of dome in a plane.

Additional undisturbed sample borings and consolidation tests were called for to accurately determine the above discussed factors at each column preparatory to the accurate plotting of consolidation curve for isolated footings, and the departure of such curve from the sine curve would determine the extent to which a continuous concrete foundation girder must distribute certain column loads to adjacent areas of lower compressibility.

Inasmuch as the foundation depth from the column base to the top of the second limestone is 22 meters, this depth will be utilized as the depth of the annular reinforced concrete foundation girder designed to even out unequal settlements by the distribution of column loads over considerable distances.

In designing the foundation girder wall, the first problem to be solved lay in the determination of the amount of settlement which would occur if each column were supported on an isolated footing and there was no stiffening ring between such footings. By considering all column loads, except that at the point under consideration, as point concentrations, Boussinesq's formula is used to secure the vertical components of stress at the point in question due to surrounding loads. At such point, the vertical component of stress may be computed by considering the load as applied uniformly at a given intensity over the rectangular concrete wall area, and the vertical stresses may be computed at various levels under the center of such area. Total settlements may be computed by adding the load at the column center to those resulting from surrounding loads and applying the consolidation curves and the boring records as to character, amount and dispersion of compressible materials. If within the range of the loads considered, the consolidation curve is assumed as a straight line, it can be said that the total settlement is composed of two parts, i. e., that part due to the load at the point in question which will vary directly with such load and that part due to surrounding loads which is fixed in magnitude and existing in the same amount irrespective of the load at the point under consideration.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by August 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

(Supplement)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . SEPTEMBER-OCTOBER, 1937

Discussion of a Paper by Carlton S. Proctor:

Recent Developments in Foundation Design*

CHARLES E. ANDREW†

The design and construction of the caissons of the West Bay Crossing of the San Francisco-Oakland Bay Bridge have been described elsewhere‡ in so much fuller detail that nothing can be added to the author's statements in this respect. However, the author has made certain statements and inferences which, for purposes of the record, should be qualified.

The application of the "dome" method for these particular caissons was first suggested to the engineers of the Bay Bridge by the late Daniel E. Moran. With this suggestion, he also submitted tentative details for incorporating these cylinders into the caissons. These tentative plans were reviewed by the Bridge Department of the San Francisco-Oakland Bay Bridge. It was found that the general conception was suitable for the foundation conditions existing at the location of the West Bay Crossing where water ranged in depths from 70 to 105 ft.

Using the type of caisson suggested by Mr. Moran, the details of the final caisson structure were developed in the San Francisco-Oakland Bay Bridge office and were compared as to cost, feasibility and safety with concrete and timber caissons of somewhat different type. These details were submitted to the Chief Engineer and to the Consulting Board and, while it was found that the cost of the caisson suggested by Mr. Moran was more expensive, it was considered a safer type because of its control features during the period of lowering from the surface and landing on mud. In consequence, this type was adopted in the final design for the West Bay Crossing.

*JOURNAL, Amer. Concrete Inst., May-June 1937; *Proceedings* Vol. 33, p. 541.
†Bridge Engineer, San Francisco-Oakland Bay Bridge.
‡*Engineering News Record*, Apr. 5 and Aug. 23, 1934 and Feb. 14 and 21, 1935.

However, in the East Bay Crossing where the deepest pier of the project was sunk 243 ft. below low water, a concrete open well caisson, which proved to be considerably cheaper, was used. The ratio in cost between the two types was 19 to 13 in favor of the concrete open-well type. The water depth in the East Bay was much less (approximately 45 ft. at Pier E-3) and the added safety factor of the type of caisson suggested by Mr. Moran probably justified the added cost.

As to patents on the type of caisson used on the West Bay Crossing, it is questionable whether or not any of the many patents now in existence, which have not yet expired, would withstand court action since long-expired patents quite fully cover the type.

At the time of receiving bids, the Engineers were notified that the design infringed certain patents. A search of the patent record disclosed many prior patents in which compressed air was to be employed for flotation. Many of these had expired. The principal patents are mentioned in the description above referred to.

AUTHOR'S CLOSURE

Mr. Andrew's discussion is inconsistent with the facts and with his own printed statements concerning the foundations for the San Francisco-Oakland Bay Bridge in the *Engineering News Record*, April 5, 1934. In this article Mr. Andrew points out that previously used types of caissons, specifically the braced timber cofferdam and false bottom types, were not practicable or adaptable for the conditions at the site, and he further indicates the reasons why the sand island method of installation was not seriously considered. He then states that "After investigating previously used methods, Daniel E. Moran, of Moran & Proctor, proposed the cylinder method. Among the advantages of this method are the stability and control which may be obtained during both the floating period and the sinking stage." Mr. Andrew further states, "This appears to be the first time this method has been employed * * *."

The facts are that the Design Department of the San Francisco Oakland Bay Bridge made several attempts to develop caisson designs for the West Bay foundations, but no such attempt on its part resulted in a safe, practicable or adequate caisson design. Acting under the terms of its contract with the Department of Public Works of the State of California, the firm of Moran & Proctor (now Moran, Proctor & Freeman) analyzed the several studies made by the Design Department and disclosed inherent weaknesses and hazards in each. The firm then developed in its New York offices the full design principles and sketches of the domed type "Moran Caisson" which was found

fully to satisfy the conditions at the site, and, in its final form, was employed in this foundation. Moran & Proctor developed a scale perspective of the dome caisson and applied for a United States Patent in the name of Daniel E. Moran. The fundamental principles involved were deemed novel and original by the United States Patent Office when it granted Patent No. 1971046 for this type of caisson. The previous methods of pneumatic floation referred to by Mr. Andrew, do not include the basic principles of construction or control employed and demonstrated in this installation.

Mr. Andrew's comparison of the cost of this type of caisson for the West Bay Crossing with other types is misleading, since no type, other than the dome caisson, was ever developed to the point where it was considered feasible or even safe, and therefore no such comparison of cost was possible.

Mr. Andrew's comparison of the caissons for the West Bay Crossing with those for the East Bay Crossing is also inconsistent. The West Bay piers required control of floating caissons to an unprecedented depth, there being a maximum of 105 ft. of open water, with soft mud below, so that caissons required design provisions to float to a depth of 120 ft. before coming to rest on a soil capable of sustaining their weights. The caissons in the East Bay Crossing were placed in a maximum depth of 40 ft. of water, offering no unusual problem in flotation or control for either the sand island or false bottom methods, the latter of which was adopted by the Contractor. Any comparison as to costs between the East Bay and the West Bay foundations is valueless. The sites and difficulties of caisson installation are not comparable.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MAY-JUNE 1937

Recent Developments in Pile Foundations*

BY MAXWELL M. UPSON†
MEMBER AMERICAN CONCRETE INSTITUTE

THE FITTING of a safe and economical substructure to the underlying earth formation constitutes a problem requiring long experience and wide knowledge. Furthermore, the problem is complicated by continuing and ever startling developments on the part of the foundation engineer. The economical method of today may be antiquated tomorrow. New tools are to be found every day.

Unlike most engineering problems, the determining factors are always varying. Broadly speaking, no two foundation soils or underlying conditions are exactly alike. Furthermore, our present methods of ascertaining the nature and relationship of this underlying material have not reached a degree of exactness that permits the generation of dependable empirical formulas. Of course, it is possible to designate sand, clay, hardpan and rock; but what each one of these materials really means in carrying power, and what is the sustaining power that results from a mixture of these so-called cardinal formations, is quite beyond the ken of our modern engineering. A clay with a 10 per cent sand may have known physical characteristics; but what happens if the fineness of this sand content is varied, or if the water content is changed, etc.?

So many variables enter the equation that a solution by modern mathematics is seemingly impossible. A concession to the possibilities of the future may be justified in the interesting and ingenious experiments that are being made in some of our engineering laboratories on soil characteristics and their actions under load. To those of the old school, the rationalization of these untold variables into an answer that is so certain and sure that the investor will gamble his million-

*Received by the Institute Nov. 10, 1936.
†President Raymond Concrete Pile Co., New York.

dollar superstructure thereon, seems far in the future. However, these are days of miraculous accomplishments. The marked achievements in soil mechanics during the last few years promise much.

It is the purpose of this paper to review and discuss somewhat in detail methods that have been provided to meet these widely varying subsoil conditions. (This subject is so involved and important that recently a World Congress of Engineers devoted almost a week to its consideration).

In general, foundations may be divided into three classifications. These are usually termed: spread, piling, and caisson. The first is now undergoing much scientific study in the laboratories already referred to, that are directing their attention to soil mechanics. New and successful methods and apparatus used in the placing of caissons make this subject worthy of special treatment. In consequence, it is not the purpose of the writer to touch upon either of these subjects. In the piling field, only the well recognized methods and the developments that are recent and somewhat outstanding will be discussed.

The mention of piling brings to the lay mind the picture of wooden piling. It is only in the last three decades that concrete and steel have usurped the use of this ancient and effective wooden construction. In soil below permanent water level, wood piles are practically everlasting. Above water level, they deteriorate rapidly. And of course they are subject to the attack of teredo, limnoria and other animal life that is found in seawater. Constantly changing water levels and the high cost of carrying foundations to an elevation sufficiently low to insure constant saturation, and the limitation of size which restricts heavy loading, limit the economic use of wood. In consequence, this discussion will be confined to the concrete and steel type of piling.

In approaching the solution of a foundation problem, safety and cost are the two determining factors. The superstructure usually involves expenditures so much in excess of the cost of the foundation, that any risk of damage due to settlement, and particularly uneven settlement, can not be justified by any attainable monetary saving in foundation cost. In consequence, the problem resolves itself to the selection of that type of permanent foundation which is safe beyond peradventure at a minimum cost figure. In other words, the question in the engineer's mind is, how can this foundation be installed with a minimum number of dollars and cents per ton of carrying capacity. The saving in cost due to shortening the time required for installation may also be an important factor.

In considering concrete piles, the two natural classifications are

precast and cast-in-place (or "in situ," as they are termed in Europe). In the former, the developments of the last few years are comparatively few. In the latter, there have been improvements and advancement that are significant to the engineer.

Again, the cast-in-place pile must be divided into two branches: first, the shell-less; and second, the shell pile.

In discussing the development of recent years in both these classifications, the early history of the concrete pile will be omitted.*

In Europe and in the Far East, the precast and shell-less pile are favored. For the engineer who fears the distortion and possible weakening of the concrete due to placing it in the ground without protection, the precast pile is used at a cost usually considerably in excess of the shell-less cast-in-place pile. This increased cost is due, (a) to the excessive amount of reinforcing steel required to handle and drive safely the precast member; and (b) more particularly to the large unavoidable expenditures resulting from the impossibility of pre-determining the required lengths. This means either serious wastage of cutoff or costly splicing. It is for these reasons that economy usually rules against the use of the precast pile. Also, the time and space required for casting, curing and storing are factors of importance.

The generation of the shell-less pile is the result of attempts to solve these economic handicaps of the precast type.

The well-known MacArthur and Simplex shell-less piles (Fig. 1 and 2) were first promoted more than 25 years ago. Because of low cost they proved popular for a time but soon lost the confidence of the engineer because of the uncertainty of securing a safe and sure carrying unit in all soils where contamination and distortion due to the driving of adjacent piles seriously weakened or severed the unprotected concrete. To meet these defects, these systems now embrace methods that provide a protecting shell.

On the other hand, the system known as the Franki (Fig. 3) recognizing the danger of the distortion and deterioration of a small pile column, have turned to the placing of piles of large diameter varying from 20 to 25 in. In certain grounds, and by the careful use of dry concrete, it has been possible to install a very satisfactory unit, which because of the size and because of the rather liberal European building requirements is loaded very much in excess of the unit loading employed in this country. The loading abroad of the 20 in. units sometimes is as much as 40 to 50 tons, and in certain soils the loading of the 25 in. units is as much as 70 to 100 tons.

*See "History and Present Status of the Concrete Pile Industry" by Charles R. Gow, Vol. 13, 1917, p. 174, *Proceedings* of the American Concrete Institute "Concrete for Foundations and Water Front Work," by Maxwell M. Upson, Vol. 20, 1924 *Proceedings* of the American Concrete Institute.

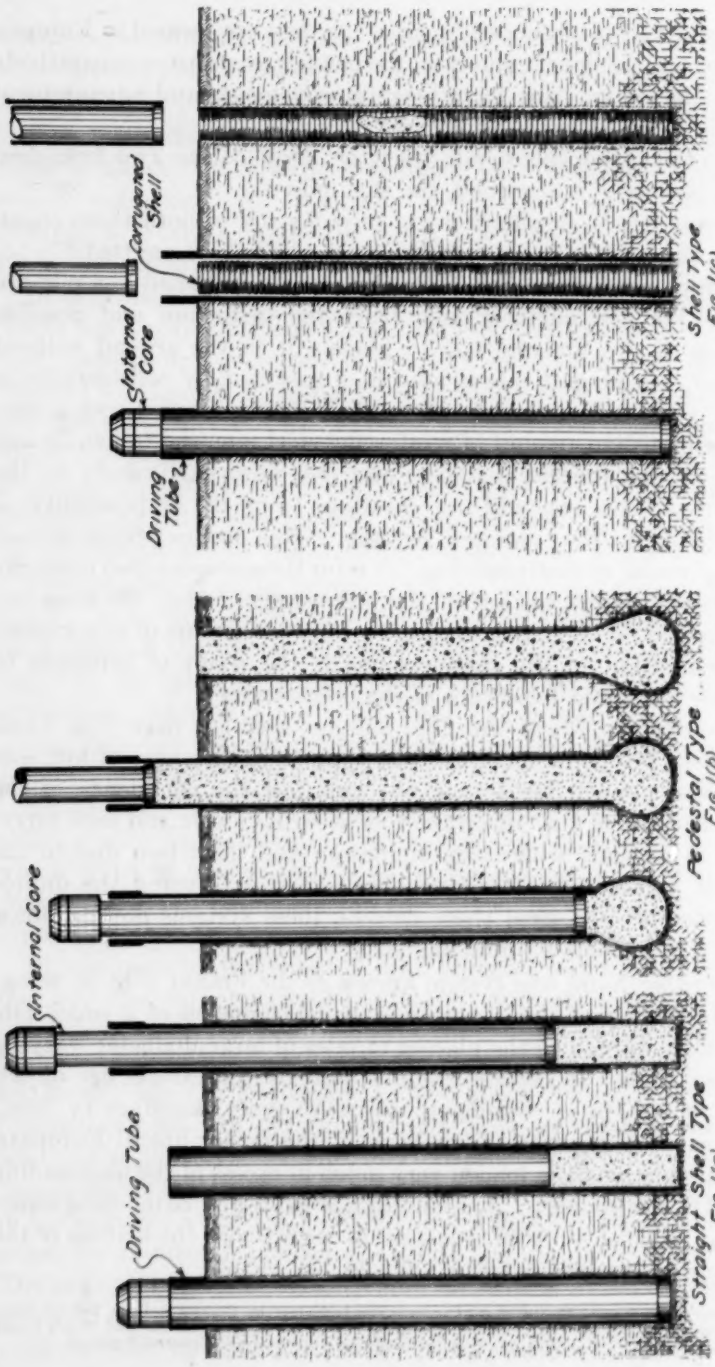


FIG. 1(C)

FIG. 1(B)

(See opposite page)

FIG. 1 (A)

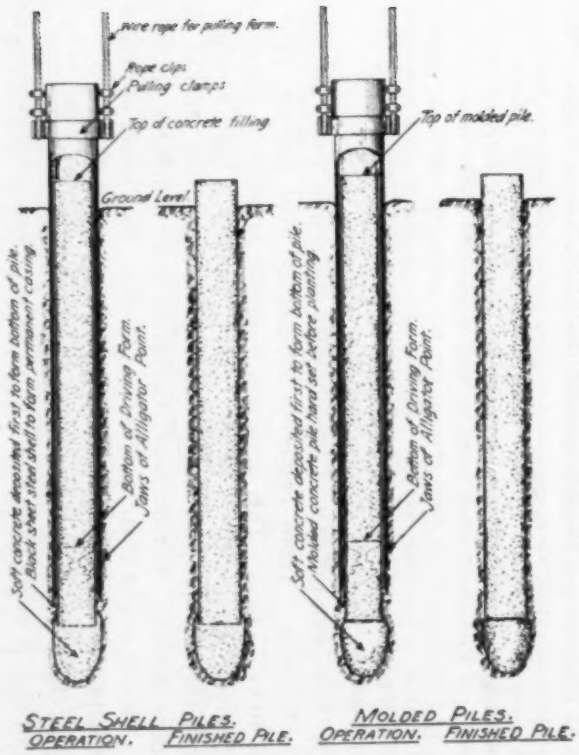


FIG. 2

FIG. 1—(ABOVE, OPPOSITE PAGE)—THREE TYPES OF PILE
(A) STRAIGHT SHAFT (B) PEDESTAL (C) SHELL (MACARTHUR COMPRESSED CONCRETE PILE)

FIG. 2—(AT LEFT)—SIMPLEX SHELL PILE AND MOLDED (SHELL-LESS) PILE

The four types of piles shown in Fig. 1 and 2 employ methods very similar in principle, namely, the driving of a tube, the filling of the tube with concrete, and then withdrawing it. The Simplex usually uses a cast iron or cast steel point on the end of the tube to keep it tight during driving and to act as a shoe at the end of the concrete. The MacArthur and Western use an internal core with a lower end fitting the tube sufficiently tight to exclude the soil during driving. Sometimes this is accomplished by driving on a flat sheet of thin metal which envelopes the point of the driving tube and core. This core is used later either to form a bulb on the end of the pile (MacArthur) or to press the concrete down during withdrawal of the shell (Western). To avoid distortion and to make sure of the exclusion of the foreign matter, corrugated shells of approximately 14-in. diameter are frequently inserted in the interior of the 16-in. tube as a form for the concrete. The advocates of this type of pile point to: (a) The economy of the shell-less pile; (b) increased point bearing area of the bulb; (c) a shell is used as assurance against distortion due to the elasticity of the soil and the pressures caused by driving adjacent piles.

The critics of this type of pile point out that: (a) The grave danger of distortion or cutting off of the concrete by ground pressures if shells are not used; (b) the excessive lengths because of uniform cross-sectional area; (c) the release of the side frictions due to the withdrawing of the driving tube; (d) the impossibility of the measuring of the carrying capacity by recording the penetration under the blow of the hammer for the reason that the side friction is reduced in withdrawing the driving tube.

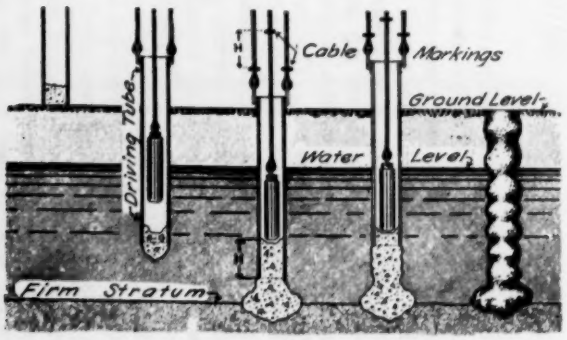


FIG. 3—STEPS IN FORMING A FRANKI PILE

The Franki pile (Fig. 3) is driven on the concrete wedged into the point of the driving tube. When firm resistance is encountered, the tube is anchored so that concrete can be driven out of the lower end to form an enlarged point. The tube is gradually filled with concrete as it is withdrawn, and at the same time the tamping is continued for the purpose of forcing the concrete into the space occupied by the walls of the tube.

The advocates of this type of pile point to: (a) The possibility of heavy loading, particularly if a hard load-carrying underlying stratum can be reached in the driving; (b) Large cross-sectional area which is less susceptible of distortion and severance by the pressures set up in the driving of adjacent piles.

The critics point out that: (a) Unless overlying soil is exceedingly soft the cross-sectional area is too great to penetrate through the intermediate strata to the hard load bearing material; (b) Frequently a lower hard stratum must be penetrated in order to carry through unstable material to a firm underlying support. This is particularly important where quicksand or other mobile soils intervene between two hard strata; (c) Since the driving consists in the tamping of the concrete and in the moving of the whole unit as an entity, no pre-test of the carrying capacity is assured; (d) Excessive loading must be used in order to justify economically the large and costly unit; (e) Such loadings are not warranted unless the underlying hard stratum is uniformly available; (f) Excessive spacing of units is essential to avoid over loading the underlying stratum unless it is rock; (g) Wide spacing involves costly footings.

The European practice of loading piles also varies from the American standards. There engineers frequently load precast piles to from 40 to 75 tons. This deviation from the practice of safety that we demand apparently develops because usually a hard stratum of gravel, clay or hardpan can be reached in most of the sites of their building centers. Furthermore, European superstructures are lighter and of lesser height than ours; in consequence, the chance of excessive overloading is much less than in this country.

Since the loading of piles necessarily hinges on the character of the ground that is being penetrated, and this may vary over every square

foot of the site, it is quite natural that the engineers in this country, in order to standardize practice, have insisted on loadings which can be predetermined by the resistance that is encountered in the driving. For that reason they have demanded that the size of the units be kept small enough to be driven by a hammer of practicable dimensions and weight.

To be certain that the carrying power of the pile conforms to the resistance indicated by the driving penetration, it is all-important that the pressures and displacements attained by the driving be maintained. This necessitates either a precast unit or a shell or pipe of sufficient strength to maintain the displacement during the driving and during the placing and setting of the concrete. Based on these all-important factors, a uniform penetration at the end of the driving provides an automatic test which assures a uniformity in the carrying power of the pile.

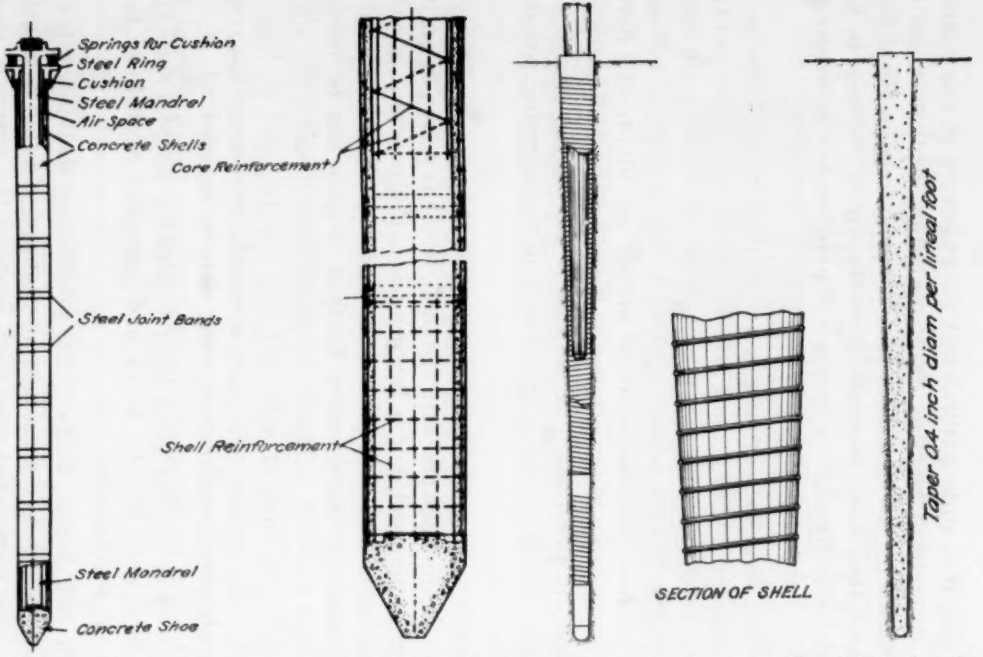
This desirable accomplishment is attained in the placing of the precast and the cast-in-place pile with the shell, which is driven in contact with the surrounding soil, but is lacking when the concrete is either poured or rammed into a displacement made by the withdrawal of the driving tube. However, when hardpan or rock can be reached through ground that will maintain its displacement, and when a concrete cross-sectional area of sufficient size can be used so that an assurance of a continuous column is attained, a comparatively safe and exceedingly cheap shell-less carrying element is provided.

That the engineering fraternity leans toward the use of a shell type of pile, is evidenced by the effort and ingenuity that have been expended in its development.

During the last decade, many of the proprietory shell-less types of pile have been modified to use a column confined in a thin corrugated steel shell. This corrugated steel shell protects the concrete, and if left unfilled until adjacent piles are driven removes the danger of distortion and severance of the column by the ground pressures that are set up by the driving. This type of pile places the principal weight responsibility on the point, which must penetrate into an underlying firm sub-stratum. If the penetrated soil is sufficiently mobile to reassume its displaced position, some assistance to the carrying power must be attained by friction.

On sites where suitable sub-stratum is available and where ground will close back around the column to give it lateral support, this unit is effective and safe. Where a pile is dependent on friction to carry the load, the release of the soil pressure by the withdrawal of the driving

FIG. 4—(LEFT)—THE ROTINOFF PILE (SEE TEXT)
FIG. 5—(RIGHT)—RAYMOND PILE



In making the pile shown in Fig. 5 a spirally reinforced sheet metal shell envelops a collapsible core. This shell is driven into the ground by means of the core to a desired penetration. It is then collapsed and withdrawn and the concrete is placed. Advocates of this pile point to (a) The conditions of driving are maintained so that the record of the penetration is an accurate measure of the safe loading of the pile. In other words, every unit is tested; (b) The shell protects the pile from distortion or the admixture of water and other materials; (c) The taper makes it possible to get carrying capacity in most sites with a length of between 60% and 75% of the straight-sided pile; (d) The encasing of the pile in a shell permits the driving on closer centers, thereby reducing the cost of the footings.

The critics of this pile point out that: (a) The shell makes the pile expensive per lineal foot; (b) The taper prevents penetrating to a deep underlying stratum.

tube must of necessity eliminate any certainty of the carrying capacity of the unit. It must be recorded that under these conditions driving resistance does not measure the load carrying capacity, for the reason that frictional conditions prevailing during driving are not preserved after the inner shell is concreted and the driving tube is withdrawn.

It is interesting to observe that the average engineer pictures the function of a pile as a unit to convey a load from a soft or unstable upper stratum to a firm load bearing sub-stratum. Records kept over thirty years, embracing several thousand piling installations, show that in many installations the point of the pile is in material of no greater bearing value than the higher strata—except for the static head. This means that the loads are carried entirely by friction. In practically all piling units, friction materially supplements the point bearing. It is evident that were this not so, the unit pressures on the piling points would far exceed loading allowances on any other than the best grade of rock.

In England, during the last ten years a concrete shell pile has been developed and successfully promoted, known as the Rotinoff Pile (Alexander Rotinoff, inventor)—Fig. 4. This has all of the advantages of precast piles, due to driving resistance and to the protection against distortion and contamination. Concrete shells from 1½ to 2½ in. in thickness, encasing a cylindrical mandrel, are driven in the ground by means of a plow boot on the point and an adjustable cushion driven on the shell. The shell section is from three to four feet long, and is provided with a sheet metal lip so that suitable caulking material may be placed to provide water and mud tight circumferential joints. The usual external diameters are 18 to 25 in. Naturally the safe loading is dictated by the diameter and by the character of soil encountered.

Another proprietory type is the Raymond Concrete Pile (Fig. 5). Its sponsors have for many years advocated the use of a shell pile of uniform taper of 0.4 in. to the foot, on the recognized theory that a taper gives a greater resistance per lineal foot of ground penetrated, and that a shell which maintains the pressure attained in driving and forms a water-tight form for the concrete provides a sure and safe pre-tested unit.

The critics of this system point out that excessive resistance caused by the taper prevents penetration to deep underlying hard stratum. To meet this criticism, and to provide a method of penetrating hard material overlying a sub-stratum, during the last five years a new type of pile has been developed—the Raymond step-taper-pile (Fig. 6). The core and shell of this pile are so sturdy that they are not

adversely affected by excessive driving. Moreover, the diameter of this pile increases only one inch in every eight feet, so that deeper penetration may be secured than would be obtainable with a pile of greater taper, assuming that both are driven to the same resistance and no impervious strata are encountered at the points.

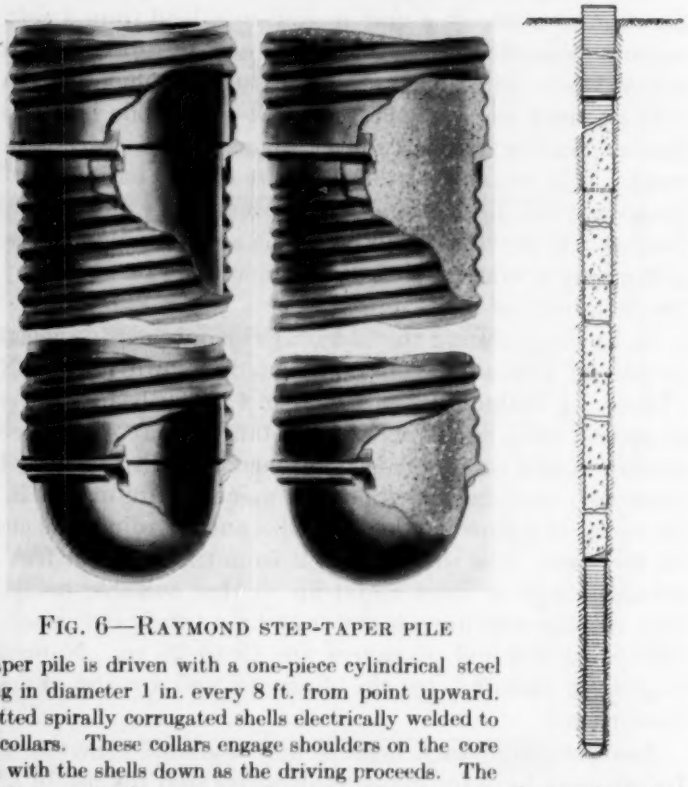


FIG. 6—RAYMOND STEP-TAPER PILE

The step-taper pile is driven with a one-piece cylindrical steel core increasing in diameter 1 in. every 8 ft. from point upward. On this are fitted spirally corrugated shells electrically welded to circular steel collars. These collars engage shoulders on the core to draw them with the shells down as the driving proceeds. The spiral corrugation permits the sections to be screwed together, thereby forming a lock and a water-tight connection. The advocates of this type of pile point to: (a) The maintenance of the conditions of driving during the filling and curing of the concrete, thereby assuring that the carrying capacity is accurately measured by driving resistance; (b) The pile is protected from distortion or contamination because of the strength and tightness of the shell; (c) Extreme lengths can be driven because of the very slight taper (56 feet maximum with light shell and indefinitely by using a pipe lower section); (d) The rather small point (10½-in.) makes it possible to penetrate the hard underlying strata which may be found in unsubstantial soils; (e) Driving on close centers is possible without distorting or cutting off the adjacent piles. The critics of this type of pile point out that: (a) Because of its lesser taper it drives to a considerably greater length; (b) The smaller point does not provide as much area on the underlying weight bearing stratum as is provided with a straight-sided pile.

Within the last few years the Union Metal Manufacturing Co., Canton, Ohio, has developed an electrically welded steel shell having the longitudinal straight plates fluted to increase the sectional modulus. Because of its shape it has been termed the fluted pile. The shell is made of not less than eleven-gauge metal, the weight of the metal depending on the character of the soil that is to be penetrated. The lower end is closed by welding on a steel point, and the upper end is reinforced with steel bands to prevent buckling under the hammer blows. This one-piece shell is driven into the ground, usually without a follower, and after the desired penetration is secured it is cut off to grade and filled with concrete. It is evident that this is a safe carrying unit, for the reason that the soil is undisturbed after attaining the final penetration, thus making it possible to predetermine the safe load.

The practical economic difficulty that faces the wide use of this system, is that the shells must be ordered of sufficient length to meet the requirement of the deepest penetration. On most sites there is a wide variation of length of pile; in consequence, the cost of waste or splicing makes the predetermination of the ultimate cost extremely difficult.

Steel pipe has an important position in the pile industry. These are driven either open-ended or closed with cast iron, malleable iron or cast steel points. In the former case, they usually penetrate to rock and the interior material is removed with either air or water jets, so that the bottoms may be inspected and the concrete deposited on the surface of the rock. When installed through all the overlying material to a full bearing on sound rock, this type of pile provides a safe and high stress carrying unit. The loading is dictated by the character of the rock on which the point rests, with suitable limitations on the diameter and thickness of the shell. Recent city building codes treat the loading of these units very specifically.

The pipe that is driven with a closed point naturally is loaded on the same basis as other accepted concrete piles. Because the pipe remains in the ground, the compression of the soil and the friction and continuity of the columns are assured.

It is natural that the steel companies should desire to find an outlet for greater tonnage in foundation work. In consequence they have promoted a steel foundation unit which is termed the H-pile.

The typical H-pile is usually of 10 or 12 in. section weighing between 40 to 60 pounds per foot. Such units usually penetrate to hardpan or rock. Since the small cross-sectional area provides very little friction per lineal foot, and since this limited area at the point must

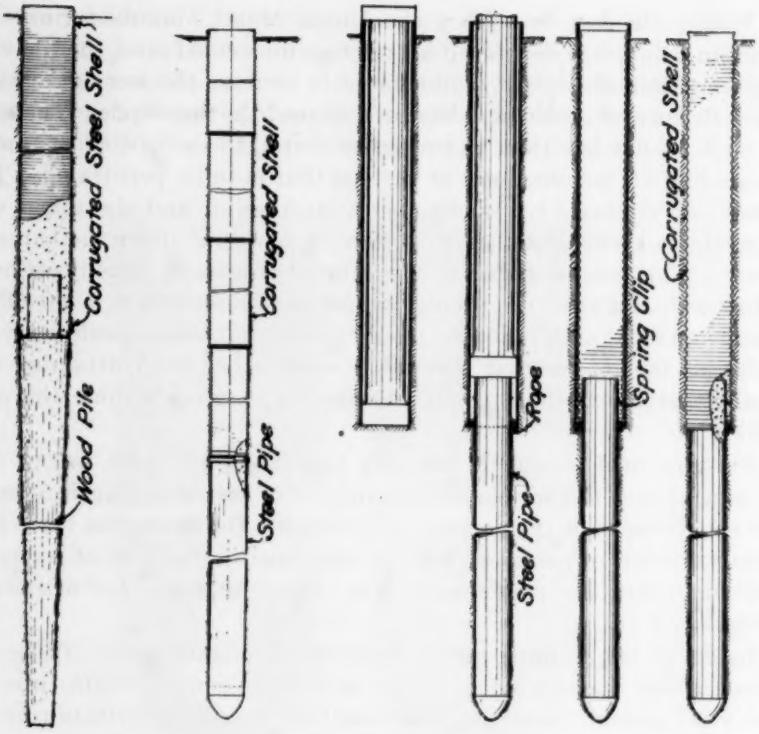


FIG. 7—(LEFT) COMPOSITE PILE—LOWER SECTION OF PIPE WITH A SUPER-IMPOSED CONCRETE SECTION

FIG. 8—(CENTER) PIPE COMPOSITE PILE—SUBSTITUTING STEEL PIPE FOR THE WOODEN LOWER SECTION (FIG. 7)

FIG. 9—(RIGHT) A VARIATION OF THE PILES IN FIG. 7 AND 8 IS SHOWN HERE WHERE THE LOWER SECTION OF A PILE IS DRIVEN THROUGH A DISPLACEMENT TUBE (SEE TEXT)

encounter extremely hard rock or hardpan in order safely to carry the load, the lengths are dictated by the location of the hard sub-stratum.

This unit can be driven through hard top strata and usually to depths very much in excess of the larger cross-section type of pile. When driven to rock of high bearing character, so that the stress between the pile and the rock is not excessive, it is an effective load carrying unit. If driven to clay or sand in which friction is an important factor, there is considerable uncertainty as to the safe loading. It is obvious that unless a very substantial and hard sub-stratum is encountered, the lengths will be materially greater than the driving

unit with the larger cross-section. Its small cross-sectional area permits it to penetrate gravel and hard strata that may overlay soft and unreliable soil formation.

The durability of unprotected steel in the ground is seriously questioned by many engineers, especially the sections above permanent water level and any portions of the pile subject to chemical or electrical disintegration.

Until recently, there have been definite limitations in the practical length of driving cast-in-place concrete piles. In the tapered pile, the reasons for this are obvious. The straight-sided tubular Simplex or MacArthur type pile is restricted in its penetration because of the excessive power required to withdraw the driving tube from the ground. Furthermore, it is expensive to make and to handle any type of pile-driving leads capable of placing piles in excess of 75 ft. high.

To meet these limitations, there have been developed various systems of driving what is termed composite piles. This term usually applies to the use of a wooden pile lower section (carried entirely below permanent water level) on which is superimposed a concrete section. It is natural in this construction that the load on the combined unit is usually limited to the bearing capacity of the lower wooden pile. In most instances, because of the normal size of timber this does not exceed a maximum of 20 tons. (Fig. 7).

As a substitute for this, and to make it possible to apply a heavier loading, a pipe composite pile has been developed. (Fig. 8). This substitutes a steel pipe for the wooden lower section. The pipe is provided with a closed cast iron or cast steel point, and is securely fastened to the shell of lighter gauge steel, which closely fits the driving mandrel. In consequence, the driving mandrel drives the pipe and pulls the shell into position.

It is obvious that a pile of this type has unlimited lengths, since the lower section can be spliced and, in sections, extended indefinitely. This type of pile has found favor in many places where friction is not safe and underlying soft strata must be penetrated in order to carry the load to deep-lying firm bearing. Standard units of this kind can readily be loaded to thirty tons or more, depending on the material penetrated and the character of the underlying firm stratum.

A similar construction is attained by driving the lower section of the pile through a displacement tube. Within this a lighter corrugated steel shell is fastened to the underlying pipe through the center of the displacement tube prior to its removal. (Fig. 9).

In approaching a foundation problem, it is all-important that adequate and carefully taken borings disclose the character and condi-

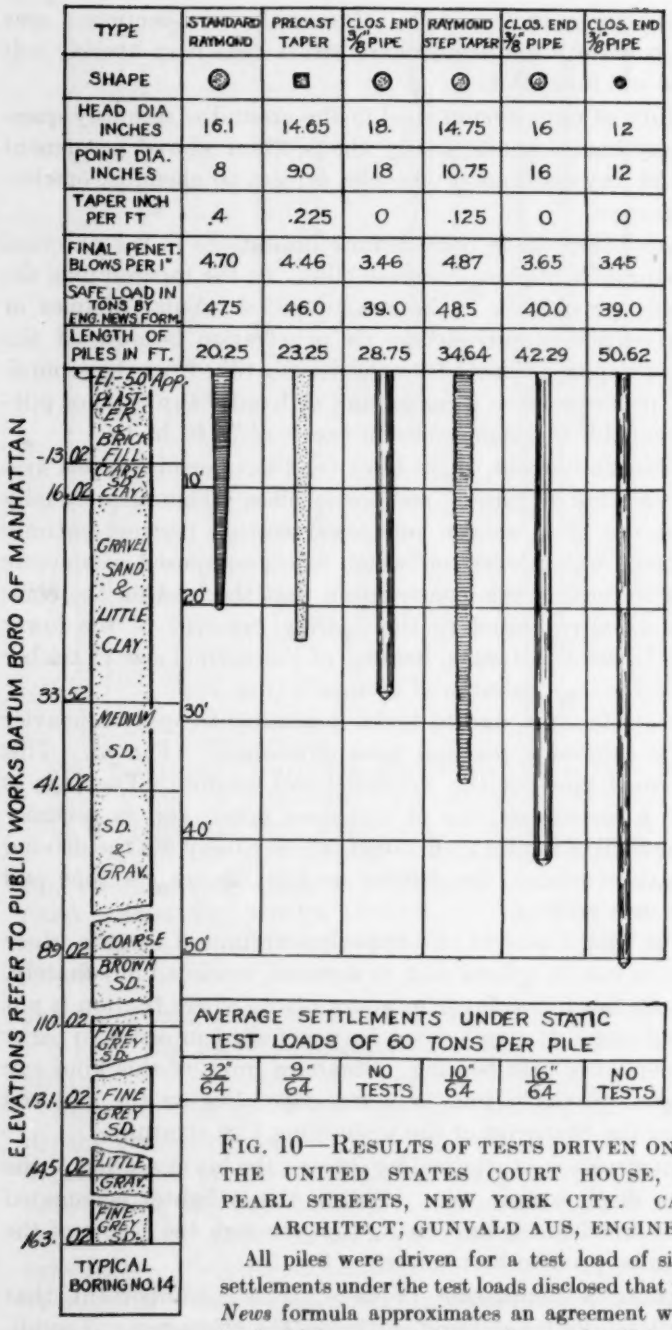


FIG. 10—RESULTS OF TESTS DRIVEN ON THE SITE OF THE UNITED STATES COURT HOUSE, DUANE AND PEARL STREETS, NEW YORK CITY. CASS GILBERT, ARCHITECT; GUNVALD AUS, ENGINEERS

All piles were driven for a test load of sixty tons. The settlements under the test loads disclosed that the *Engineering News* formula approximates an agreement with the working load which was superimposed, namely, 35 tons.

tion of the underlying material. With this information, designs may be prepared and approximate estimates of the cost of a safe foundation can be assembled.

If the loads are heavily concentrated and rock or good hardpan can be reached comparatively near the surface, and if no water problem is involved in excavation, caissons usually prove the most economical. If any of the above factors militate against the use of caissons, consideration must be given to piles.

In case water level is close to the cut-off elevation and there is no danger of this water level being lowered at a later date, wood piles may prove economical, although this is not axiomatic due to the fact that the limitation in the loading of wood piles requires almost double the number and, in consequence, more than double the size of the footings.

If concrete piles are to be employed, the predetermining of the length is all-important in order to ascertain the cost. If hardpan or rock are at hand, the lengths of the different types of pile may be approximated with fair accuracy. If the piles are to carry their load by friction, or the underlying harder stratum is not suitable for heavy point bearing, the length will depend on the character of pile used.

A large steel company in a river valley of deep sedimentary deposits contemplated putting a new open hearth furnace on two 60-foot wood piles spliced together, thereby securing a pentration of more than 120 feet. Ultimately, these furnaces were carried on heavily tapered piles averaging 22 feet long. This is an extreme illustration of how all-important is the selection of the type of pile to meet the underground conditions involved in the problem. (Fig. 10).

The use of precast concrete sheet piles for providing water-tight walls is a development of recent years, and was brought about by most unusual conditions.

Some five or six years ago, the oil companies of western Venezuela discovered a very rapid deterioration of steel sheet piling and steel members exposed to the water of Lake Maracaibo. After much research, they concluded that this deterioration (which amounted in some instances to the eating away of a half inch of steel within a year) was due to bacteria in the water which apparently secreted acid. In consequence, a substitute water-tight sheet wall had to be found to take the place of sheet steel piling which had been used for dike construction. This dike was necessitated by the subsidence of the earth due to the removal of the deposits of underlying oil. At the time the work was contemplated, the subsidence was continuing at the rate of approximately a foot a year. For that reason it was necessary

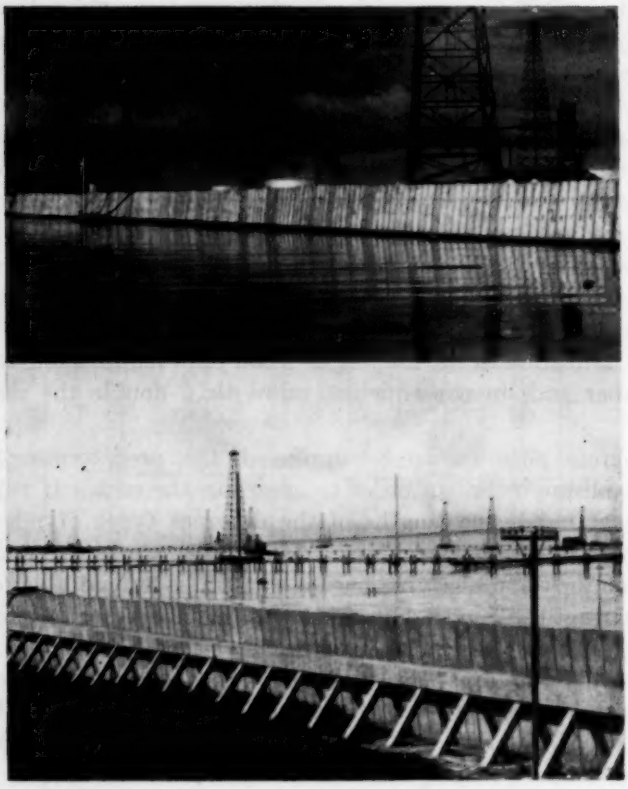


FIG. 11 AND 12—A REINFORCED CONCRETE SHEET AND PILE SEA WALL ON LAKE MARACAIBO, VENEZUELA

to design a structure of considerable height, and to provide for a superimposed increase if necessary.

To meet this requirement, a reinforced concrete sheet pile 12 in. thick, 24 in. wide and approximately 30 ft. long was used. This penetrated through the soft top material into underlying impervious clay, and extended up some 12 feet above the water surface. (Fig. 11, 12 and 13).

By the use of a V-type joint, with provision for grouting, it was possible to make this wall absolutely water-tight. Expansion joints were provided at 50-foot intervals. Approximately three miles of this was built in 1932. It has proved to be not only-water-tight but of sufficient strength to withstand the wave action of Lake Maracaibo, 50 miles wide at this point.

The many structures that are standing today after 20 to 25 years exposure between high and low water, and to the trying actions of seawater, chemicals and frost, demonstrate that concrete can be made to withstand these actions. On the other hand, frequent failures have caused the engineer to feel that concrete for such service should have a waterproof surface.

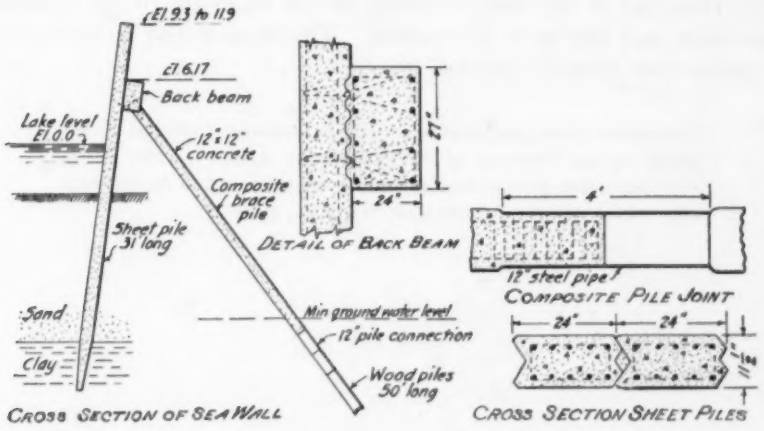


FIG. 13—DETAIL OF SEAWALL SHOWN IN FIG. 11 AND 12

To meet this demand, several methods and materials have been developed, which vary from a surfacing coating of asphalt, coal tar or heavy oil, to a complete vacuum creosoting treatment. The latter, although seemingly effective, is considered unnecessarily expensive, for the reason that it treats the entire length of the pile while the section between high and low water usually is the only portion that is adversely affected. Furthermore, in the opinion of many engineers, the internal stresses in the pile caused by the heat of this process weaken the structure of the unit.

If an asphalt impregnated surface is desired, a more economical and extremely effective method consists of casting the exposed portion of the pile in a form made of thin slabs of asphalt impregated concrete. These slabs adhere to the concrete and form a waterproof exterior.

A very effective method of sealing the concrete and protecting the surface, is by the use of either a thin linseed oil or a 20 per cent solution of ammonium stearate. This should be applied with a spray on the dry surface. It is soaked into the pores and acts as a seal. After this has thoroughly penetrated, a hot coal tar application containing about

10 per cent of china oil will adhere and provide enough elasticity to take the variations in temperature which are normally encountered.

If the water carries moving ice or other destructive flotsam, a box of thin creosoted planking may be strapped to the exposed surfaces, which can be replaced cheaply when necessary.

It must be remembered that the most essential requisite for impervious concrete (in addition to providing proper materials and constant proportioning) is the careful curing before exposure to the alternate saturation and drying of tide action. This means that members used in this service must be precast.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by August 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MAY-JUNE 1937

Properties of Cements and Concretes Containing Fly Ash*

By Raymond E. Davis[1], Roy W. Carlson[2], J. W. Kelly[3], and Harmer E. Davis[4]

MEMBERS AMERICAN CONCRETE INSTITUTE

SUMMARY

This paper presents the results of tests to determine the effect of using fly ash as a replacement for portland cement upon the properties of mortars and concretes.

In the test program were included fly ashes from 15 different sources and portland cements of seven compositions. Fly ashes in percentages up to 50 were blended with the portland cements. Other variables under investigation included chemical composition and fineness of fly ash and chemical composition and fineness of portland cement. The properties investigated included strength, elasticity, volume change, durability as indicated by resistance to freezing and thawing and by resistance to the action of sodium sulfate, plastic flow, and heat of hydration.

The results of the investigation indicate that fly ashes of moderately low carbon content and moderately high fineness exhibit a high degree of puzzolanic activity as compared with most natural puzzolans. When such fly ashes are used in moderate percentages as a replacement for portland cement, there can be produced concretes which exhibit qualities equal and in some respects superior to those exhibited by concretes containing corresponding portland cement without fly ash.

As compared with concretes containing portland cements, corresponding concretes containing properly constituted fly-ash cements exhibit (1) about the same water requirement to produce a given con-

*Presented by Raymond E. Davis at the 33rd Annual Convention, Amer. Concrete Inst., New York, February, 23-26, 1937.

[1]Professor of Civil Engineering, University of California, Berkeley.
[2]Associate Professor of Civil Engineering, Massachusetts Institute of Technology, Cambridge.
[3]Research Engineer, Engineering Materials Laboratory, University of California, Berkeley.
[4]Assistant Professor of Civil Engineering, University of California, Berkeley.

sistency; (2) somewhat lower compressive strength at the early ages but substantially higher compressive strength at the later ages under normal conditions of moist curing (70° F.); (3) compressive strengths which are substantially higher even at the relatively early age of 28 days when the temperature of curing is 100° F.; (4) shrinkage, for sections of normal thickness and normal drying conditions, which is likely to be no more and may be less; (5) lower heat of hydration; (6) about the same or somewhat less resistance to freezing and thawing; and (7) greater resistance to sulfate action.

In general the performance of fly-ash cements appears to be most satisfactory when the portland-cement constituent is of normal fineness and of normal or high-lime composition.

For ordinary construction it appears that fly ashes of moderately low carbon content and moderately high fineness may be employed as replacements for portland cement in percentages up to 30 without in any way impairing the qualities of the concrete. For heavy concrete construction it appears that such fly ashes may be employed as replacements for portland cements in percentages as high as 50, with the advantages of substantially lower temperature rise due to hydration of the cement and of higher ultimate compressive strength.

Suggested specification requirements for fly ash include limitations on the chemical composition, fineness, water requirement, and activity in the presence of lime.

INTRODUCTION

The residue from the burning of powdered coal is commonly designated as "fly ash." As the powdered coal passes through the zone of high temperature within the boiler, the carbon is burned and the residue remains in suspension in the form of fused particles. These particles, still in suspension, are quickly carried to a zone of comparatively low temperature where they solidify to form the fly ash; for some fly ashes many of the particles are of spherical shape. The ash, which if discharged from the stack into the atmosphere would constitute a dust nuisance, is caught by precipitators.

Between fly ashes derived from various coals there exist differences in chemical composition, but the principal compounds are silica, alumina, iron oxide, and lime. In all cases the compound present in the largest percentage is silica, and in all cases there is present in the ash some carbon, but under proper plant conditions it appears that the carbon content will be low. Fly ashes from most plants burning powdered coal are characterized by their extreme fineness, the surface area per unit of weight being on the average perhaps twice that of a normal portland cement.

It is known that certain finely divided siliceous materials of volcanic origin when mixed with lime produce a hydraulic cement. The pozzuolana of ancient Rome was a volcanic ash possessing such characteristics. It is generally recognized that, during the process of hydration of portland cement, lime is liberated. In exposed concrete structures, this free lime is frequently leached from the concrete and appears on the surface in the form of efflorescence. In the case of concretes subjected to the action of percolating water which is evaporated as it reaches the surface, there frequently appear incrustations of lime carbonate having a thickness of an inch or more.

Portland-puzzolan cements are composed of a mixture of portland cement and a siliceous material which in the presence of water possesses the property of combining with the lime liberated during the hydration of the portland-cement constituent, *i.e.*, a material which posseses puzzolanic activity. It is usually considered that the principal compound formed by reaction between the puzzolan and the lime liberated by the portland cement is monocalcium silicate. This compound, which is insoluble in water, forms slowly and contributes to a long-continued gain in strength of concrete as well as to watertightness.

Natural materials which exhibit puzzolanic activity to a marked degree are pumicites, trasses, tuffs, diatomaceous earths, and shales. These materials are either interground with the portland-cement clinker during the process of manufacture or used as a replacement for cement, being added in a finely divided state either at the concrete mixer or in a blending plant. The proportion of natural puzzolan which has been used as a replacement for cement varies considerably, but perhaps averages 25 percent.

Portland-puzzolan cements containing suitable natural puzzolans appear to possess certain advantages over normal portland cement. Concretes in which they are employed are generally observed to possess greater workability, with less tendency towards segregation of the aggregates, than portland-cement concretes. Although at the early ages the compressive strength of concrete is lower than that of corresponding concrete containing normal portland cement, there is generally a long-continued gain in strength, so that at the later ages the compressive strength may exceed that of a concrete containing normal portland cement. Also, concretes containing portland-puzzolan cements are characterized by a high degree of impermeability and by excellent resistance to the action of aggressive waters. However, in general, concretes containing portland-puzzolan cements require the long-continued presence of water for satisfactory development of strength; and concretes containing some portland-puzzolan

cements shrink considerably more upon drying than do concretes containing corresponding portland cements.

Since the conditions under which fly ash is produced are similar to those under which volcanic ash is produced, it was considered possible that fly ash of reasonably low carbon content and high fineness might prove to be an artificial puzzolanic material of satisfactory activity and that, because of the generally spherical shape of the particles and their extreme fineness, through its use there might be produced a portland-puzzolan cement of superior properties.

In order to investigate these possibilities, quantities of fly ash were obtained from 15 power plants in the East and Middle West (Table 1),

TABLE 1—SOURCE OF PUZZOLANS

Puzzolan	Source	Location
Chicago fly ash	Chicago District Electric Generating Corp.	State Line Plant, Hammond, Ind.
Cleveland fly ash	Cleveland Electric Illuminating Co.	Avon Plant, Cleveland, Ohio
Indiana fly ash	Northern Indiana Public Service Co.	Michigan City, Ind.
West Penn fly ash	West Penn Power Co.	Springdale, Pa.
Union Electric fly ash	Union Electric Light and Power Co.	Cahokia Station, St. Louis, Mo.
Detroit fly ash	Detroit Edison Co.	Trenton Channel Plant, Detroit, Mich.
Duquesne fly ash	Duquesne Light Co.	Colfax Station, Cheswick, Pa.
Long Island fly ash	Long Island Lighting Co.	Glen Cove, Long Island, N. Y.
Potomac fly ash	Potomac Electric Power Co.	Buzzard Point Station, Washington, D. C.
New York Edison fly ash	New York Edison Co.	14th St. (East River) Station, New York, N. Y.
Cos Cob fly ash	N. Y., N. H., & Hartford R. R.	Cos Cob, Conn.
Stamford fly ash	Stamford Gas & Electric Co.	Stamford, Conn.
New York Steam fly ash	New York Steam Corp.	Kips Bay Station, New York, N. Y.
Hell Gate fly ash	New York Edison Co.	Hell Gate Station, New York, N. Y.
Boston fly ash No. 1	Boston Elevated Railway Co.	Boston, Mass.
Boston fly ash No. 2	Boston Elevated Railway Co.	Boston, Mass.
Pumicite	——	Fresno Co., Calif.
Limestone dust	——	Colton, Calif.

and these fly ashes, together with portland cements, were utilized in the manufacture of portland-puzzolan cements varying from one another as to chemical composition and fineness of the fly ash and of the portland-cement constituent, as to the proportion of fly ash (10 to 50 percent by weight), and as to the method of blending (by admixing

or by intergrinding). In order to make the comparisons more complete, cements similar to the fly-ash cements were prepared using a natural puzzolan (pumicite) and an inert material (limestone dust).

The cements were utilized in three series of tests to determine strength, elasticity, volume changes, heat of hydration, resistance to the action of freezing and thawing, resistance to the action of sodium sulfate, etc. The total number of cements was 81, the total number of specimens was approximately 5000, and the total number of tests and observations exceeded 11,000.

General Investigation

The first of the three series of tests was undertaken in order to determine the properties of cements, mortars, and concretes in which fly ashes having a wide range in composition and fineness were used as a replacement for portland cement. While the majority of tests were made on fly-ash cements containing a portland cement of a moderate-lime, low-alumina composition and of a moderately high degree of fineness, there was also investigated the effect upon various properties of composition and fineness of the portland cement, the proportion of fly ash, and the method of blending.

The distinguishing characteristics of the portland cements were as shown in Table 2:

TABLE 2—TYPES OF PORTLAND CEMENT

Portland Cement	Tricalcium Silicate, Per Cent	Tricalcium Aluminate, Per Cent	Specific Surface, sq. cm. per gram
High-early-strength	60	8	2220
Normal	53	12	1590
General-purpose	50	6	2220
			1600
			1170
Low-heat	42	4	1600

In general the fly-ash cements contained 20 percent of fly ash, but several cements contained 30 percent and one cement contained 10 percent of fly ash. For most of the cements, the fly ash and portland cement were blended by simply admixing at the mixer, but for four of the fly-ash cements the materials were interground.

The general scope of the investigation is indicated by Tables 3 and 4. The results of tests to determine selected important properties of all cements are shown in Table 3, and the results of tests to determine additional properties of selected cements are shown in Table 4.

TABLE 3—RESULTS OF TESTS ON PORTLAND-PUZZOLAN CEMENTS CONTAINING
(Table continued opposite page)

Cement				Neat Paste[2]			Standard Mortar	
% of Puzzolan	Puzzolan	% Carbon in Fly Ash	Specific Surface, sq. cm. per gram[1]	Norm. Cons., % water by wt.	Time of Setting, hr.:min. (Gillmore)		Water-Cement Ratio, by wt.	Tensile Strength, p.s.i. (Briquets)[3]
					Init.	Final		28 da.
1	2	3	4	5	6	7	8	9
0	None	——	1600	22.5	2:15	4:00	0.41	365
10	Chicago fly ash	1.1	1760	22.5	2:45	4:40	0.41	390
20	Chicago fly ash	1.1	1920	22.2	3:00	5:00	0.41	350
	Cleveland fly ash	1.1	1770	23.3	3:40	5:00	0.42	430
	Indiana fly ash	3.3	1800	22.8	3:10	5:00	0.41	390
	West Penn fly ash	6.4	1900	24.5	3:05	5:10	0.42	375
	Union Elec. fly ash	7.4	1810	22.5	3:40	5:45	0.41	380
	Detroit fly ash	7.5	1890	25.0	3:25	5:35	0.43	370
	Duquesne fly ash	9.3	1800	26.5	3:30	4:45	0.44	350
	Long Island fly ash	10.4	2040	28.0	4:10	6:30	0.45	370
	Potomac fly ash	11.9	1760	25.5	3:25	5:20	0.43	330
	N. Y. Edison fly ash	13.6	1770	29.5	3:25	5:25	0.46	415
	Cos Cob fly ash	13.9	1910	28.5	3:45	6:00	0.45	325
	Stamford fly ash	15.7	1880	27.5	4:55	6:45	0.44	330
	N. Y. Steam fly ash	16.6	1800	27.5	3:45	5:50	0.44	355
	Hell Gate fly ash	19.3	1670	30.0	3:00	5:25	0.46	335
	Boston fly ash No. 1	26.4	1580	29.0	3:30	5:20	0.45	310
	Boston fly ash No. 2	30.2	1620	30.0	3:50	5:45	0.46	335
	Average for 16 fly ashes	12.1	1810	26.4	3:35	5:30	0.44	360
	Pumicite	——	1940	26.0	3:10	4:40	0.43	375
	Limestone dust	——	2130	22.5	1:55	3:15	0.41	350
30	Chicago fly ash	1.1	2090	22.0	3:20	6:20	0.41	410
	West Penn fly ash	6.4	2040	24.0	3:30	6:15	0.42	330
	Long Island fly ash	10.4	2260	30.0	4:25	6:45	0.46	365
	Potomac fly ash	11.9	1840	26.5	3:35	6:05	0.44	315
	Limestone dust	——	2390	23.0	2:00	3:25	0.41	330

[1]Specific surface of portland-puzzolan cements computed from specific surfaces of separate materials.
[2]All cements sound in standard steam test on neat-cement pat.
[3]Standard curing.

Tests on High-Early-Strength Portland-Puzzolan Cements

It is a general characteristic of portland-puzzolan cements that, compared with corresponding portland cements, they exhibit relatively low strength at the early ages. The results of the first series of tests indicated, however, that certain of the fly ashes possessed a high degree of activity even at the early ages, and that there existed the possibility of the development of a portland-puzzolan cement having high-early-strength characteristics even with replacements as high as 20 or 30 percent. Certain advantages would appear to result from the use of a portland-puzzolan cement having rapid-hardening characteristics: (1) reductions in cost of manufacture and grinding resulting from replacement of portland cement by an active puzzolan of an initially high degree of fineness, and (2) the long-continued gain in strength characteristic of portland-puzzolan cements in contrast

PUZZOLAN ADMIXED WITH GENERAL-PURPOSE PORTLAND CEMENT

(Table continued from opposite page)

Concrete[4]										
Water-Cement Ratio, by wt.	Compressive Strength, p.s.i. (3 by 6-in. Cylinders)[3]				Init. Tan. Modulus of Elasticity, mill. p.s.i.[3]	Length Change, millionths (1½ by 1½ by 12-in. Bars)[3]				
						Exp. in Fog[5]	Net Contraction (Air Storage)[5]			
	7 da.	28 da.	3 mo.	1 yr.	1 yr.	28 da.	35 da.	56 da.	4 mo.	1 yr.
10	11	12	13	14	15	16	17	18	19	20
0.43	4040	5310	5790	6810	6.2	25	265	380	440	475
0.41	4010	5260	6110	7500	6.7	25	285	410	480	515
0.40	3760	5190	6230	7610	6.8	30	260	385	455	490
0.42	3720	4510	5700	6760	6.2	25	285	445	510	540
0.41	3540	5010	6330	7190	6.4	40	285	440	500	525
0.43	3370	4910	6260	7450	6.4	25	280	435	505	530
0.45	3330	4510	5620	6410	6.1	30	270	400	455	500
0.44	3170	4790	5870	6690	6.2	40	325	510	580	605
0.48	2890	4220	5200	6130	5.7	30	300	445	505	555
0.46	3220	4910	6390	7080	6.4	20	325	465	535	570
0.45	3270	4770	5830	6840	6.1	30	280	430	505	540
0.49	2940	4310	5360	6240	6.0	25	310	520	580	610
0.49	2830	3770	5400	6540	6.3	30	325	505	570	605
0.47	2980	4100	5480	6550	6.2	25	320	495	560	600
0.49	2820	4350	5330	5930	5.8	30	325	485	545	585
0.49	2810	3900	4940	5720	5.8	35	295	495	605	650
0.50	2620	3950	4970	5840	5.8	40	310	485	520	600
0.52	2640	3830	4700	5330	5.8	35	305	465	525	555
0.46	3090	4440	5600	6520	6.1	30	300	465	530	565
0.44	3180	4590	5200	6160	6.2	30	340	505	575	620
0.43	3600	4520	5150	6130	5.9	15	285	455	525	545
0.40	3210	4970	6400	7770	6.8	15	260	380	460	495
0.43	2850	4400	5900	7390	6.4	30	290	425	490	515
0.48	2640	4250	5800	6730	5.8	20	320	465	530	565
0.46	2590	3960	5250	6360	5.9	30	305	500	585	630
0.43	3120	4030	4680	5320	5.8	20	275	465	535	555

[4]Cement-aggregate ratio = 1:5.6 by wt.; 0 to ¾-in. aggregate; flow 60 per cent.
[3]All values referred to length at 3 days.
[5]Standard curing for 28 days, then storage at 70° F. in air of 50% relative humidity.

with the tendency of many high-early-strength portland cements to show retrogression in strength at the later ages. A probable disadvantage is the necessity for adequate moist curing in order to develop and maintain puzzolanic action.

Accordingly, a second series of tests was undertaken, in which two groups of cements were employed. For one group of cements a fly ash of low carbon content and moderately high fineness was interground with a portland cement (50 percent 3CS, 8 percent 3CA) in various percentages ranging from 10 to 40. For the second group of cements, a fly ash of low carbon content and moderately low fineness was interground with a portland cement (56 percent 3CS, 8 percent 3CA) in various percentages ranging from 10 to 40. For the latter group a natural puzzolan (pumicite) was also employed. The general features of this investigation are shown in Table 9, in which are given the results of the tests.

TABLE 4—RESULTS OF ADDITIONAL TESTS ON SELECTED PORTLAND-PUZZOLAN
(Table continued opposite page)

Fly Ash in Portland-Puzzolan Cement	% Carbon in Fly Ash	Specific Surface of Cement, sq. cm. per gram[1]	Standard Mortar								
			Water-Cement Ratio, by wt.	Tensile Strength, p.s.i. (Briquets)							
				Standard Curing[2]						Air Storage[2,3]	
				1 da.	3 da.	7 da.	28 da.	3 mo.	1 yr.	3 mo.	1 yr.
1	2	3	4	5	6	7	8	9	10	11	12
None	——	1600	0.41	140	275	310	365	395	425	430	305
Chicago	1.1	1920	0.41	110	245	285	350	515	530	480	415
West Penn	6.4	1900	0.42	105	225	295	375	425	480	415	375
Long Island	10.4	2040	0.45	75	220	265	370	465	520	435	355
Potomac	11.9	1760	0.43	115	225	280	330	420	475	450	430
Hell Gate	19.3	1670	0.46	120	225	260	335	415	465	405	375
Avg. for 5 fly ashes	9.8	1860	0.43	105	230	275	350	450	495	435	390

Fly Ash in Portland-Puzzolan Cement	Concrete (3 by 6-in. Cylinders)[7]									
	Water-Cement Ratio, by wt.	Resistance to Sodium Sulfate[11]					Resist. to Freezing and Thawing			
		Appearance Rating[12]			Compressive Str. at 6 Mo.		Loss in Weight, %	Appearance Rating[12]	Compr. Str. at 6 Mo.	
		4 mo.	5 mo.	6 mo.	p.s.i.	Ratio[13]		6 mo.	p.s.i.	Ratio[13]
25	26	27	28	29	30	31	32	33	34	35
None	0.43	10	9	7	5560	0.88	0.2	9	6050	0.96
Chicago	0.40	10	10	8	7120	1.03	0.2	9	6780	0.98
West Penn	0.43	10	9	7	6580	0.97	0.1	9	5060	0.75
Long Island	0.46	10	10	8	6400	0.96	0.0	9	5490	0.83
Potomac	0.45	9	8	5	5830	0.91	0.1	9	4920	0.77
Hell Gate	0.49	10	9	6	5060	0.93	0.0	9	3560	0.65
Avg. for 5 fly ashes	0.45	10	9	7	6200	0.96	0.1	9	5160	0.80

[1]Specific surface of portland-puzzolan cements computed from specific surface of separate materials.
[2]Standard curing for 28 days, then storage at 70° F. in air of 50% R. H.
[3]Standard curing; specimens tested damp.
[4]Specimens standard-cured for 5 months, then subjected to thirty 24-hr. cycles of treatment; saturated.
[5]Ratio of compressive strength of standard-cured concrete at age indicated to heat of hydration of neat-cement paste at 28 days.
[6]Contraction referred to length at 28 days.
[7]1 Part of cement by weight to 5.6 parts of 0 to ¾-in. local gravel; flow 60 per cent.

Portland-Puzzolan Cements Containing High Replacements of Puzzolan

The results of the first series of tests indicated that through the use of a high replacement there might be produced a fly-ash cement of very low heat of hydration, particularly suitable for mass-concrete construction. To investigate this possibility there were employed two fly ashes of low carbon content and (for comparison) a pumicite, in replacements of 30 and 50 percent. The portland cement was commercially ground from a general-purpose or modified type of clinker. The portland cements and puzzolans were blended by being admixed

CEMENTS CONTAINING 20 PER CENT OF FLY ASH ADMIXED WITH GENERAL-PURPOSE PORTLAND CEMENT

(Table continued from opposite page)

Water-Cement Ratio by wt.	Concrete (3 by 6-in. Cylinders)[7]								Strength-Heat Ratio, p.s.i. per cal. per gram[8]		
	Compressive Strength, p.s.i.										
	Standard Curing[4]					70-100° F. Curing[5]					
	7 da.	28 da.	3 mo.	6 mo.	1 yr.	7 da.	28 da.	3 mo.	28 da.	3 mo.	1 yr.
13	14	15	16	17	18	19	20	21	22	23	24
0.43	4040	5310	5790	6310	6810	4300	5320	5790	63	69	81
0.40	3760	5190	6230	6950	7610	4200	6080	6750	69	83	101
0.43	3370	4910	6260	6800	7450	3850	6020	6380	61	78	93
0.46	3320	4910	6390	6650	7080	3700	5660	6110	62	81	90
0.45	3270	4770	5830	6420	6840	3800	5610	5880	60	74	86
0.49	2810	3900	4940	5480	5720	3220	4700	5140	51	65	75
0.45	3290	4740	5930	6460	6940	3750	5610	6050	61	76	89

Concrete (6-in. Cubes)[7]									Neat Paste			
Contraction, millionths (Parallel to Exposed Face and at Given Depth; All Other Faces Sealed)[3,6]									Heat of Hydration, Calories per gram (After 15 Min.)[10]			
¼ In. from Face			2 In. from Face			5¾ In. from Face						
35 da.	56 da.	1 yr.	35 da.	56 da.	1 yr.	35 da.	56 da.	1 yr.	1 da.	3 da.	7 da.	28 da.
36	37	38	39	40	41	42	43	44	45	46	47	48
180	255	515	35	65	290	0	−10	95	33	55	72	84
160	240	480	20	50	245	5	− 5	30	29	51	62	75
155	290	570	30	90	310	5	−10	45	29	47	64	80
155	250	445	30	65	270	−5	−10	35	29	50	67	79
170	300	540	45	105	355	10	0	75	30	49	64	79
145	295	640	30	75	355	15	−10	60	32	51	64	76
155	275	535	30	75	305	5	− 5	50	30	50	64	78

[4]Curing in sealed cans at 70° F. for 1 day, then at 100° F. until 1 day prior to test, and then at 70° F.; specimens tested damp.
[5]Specimens tested dry.
[10]Water-cement ratio 0.40 by wt.; Carlson vane calorimeter; control temperature 70°F.
[11]Specimens standard-cured for 28 days, then immersed in 10% solution of Na_2SO_4.
[12]Rating scale: 10 perfect, 9 excellent, 7 good, 6 intermediate, 3 poor, 1 very poor.
[13]Ratio of compressive strength of treated specimens to strength of untreated specimens standard-cured up to same age of test.

in a laboratory batch mill equipped with wooden vanes. In addition to the determination of strength of concretes made and tested under standard conditions, there was determined the strength of concretes under an accelerated curing cycle, designed to simulate mass-curing conditions, inasmuch as the response of a portland-puzzolan cement to mass curing may be much more favorable than to ordinary conditions.

The principal details of this series of tests are indicated by Table 10 which gives the results of the tests.

PROPERTIES OF THE FLY ASHES

Chemical Composition of Fly Ashes

Except as regards the percentage of carbon, the differences in chemical composition of the various fly ashes under investigation were not large, even though the fly ashes were derived from widely scattered coal deposits. Table 5 shows the range and average values of the principal oxides present in two groups of fly ashes—one of low carbon content and the other of high carbon content. (The complete chemical analysis of the fly ashes is shown in Table 6.)

TABLE 5—CHEMICAL COMPOSITION OF FLY ASHES, BY GROUPS

Group		Percentage					
		C	SiO_2	SiO_2 in Non-carbon Portion	Fe_2O_3	Al_2O_3	CaO
3 Fly ashes of low carbon content	Range	1–3	42–44	43–45	12–19	16–28	3–7
	Avg.	2	43	44	16	22	5
5 Fly ashes of high carbon content	Range	16–30	28–38	40–45	7–14	18–25	1–8
	Avg.	22	33	42	10	22	4

It will be observed that the carbon content varied widely, ranging from 1 to 30 percent. For one fly ash not herein considered the carbon content was 54 percent. The silica content of the fly ashes as a whole ranged from 28 to 44 percent, but for the non-carbon portion of the fly ashes the silica content fell within the narrower limits of 40 to 45 percent. The lime content ranged from 1 to 8 percent. The percentages of the alkalies and of sulfur and its compounds were small.

Although the effects of differences in chemical composition other than carbon content are not discussed herein, it may be stated that as among the fly ashes of this investigation, except for carbon content the differences in chemical composition of fly ash appear to have no marked effect upon the properties of concretes.

Fineness of Fly Ashes

In general, the fly ashes of low carbon content were considerably finer than those of high carbon content, as indicated by the average values for two groups of fly ashes in Table 7. (Data regarding the fineness of individual fly ashes are shown in Table 6.)

The coarseness of the high-carbon fly ashes was due in part to the fact that the carbon particles were relatively large.

In the last column of Table 7, the fineness is expressed in terms of specific surface, which is a calculated value of the surface area of the particles per gram of fly ash. The calculations are based upon changes in density of a column of water in which a given quantity of fly ash

was dispersed and then allowed to settle; the changes in density were observed with a sensitive hydrometer.* Although the calculated values may be open to question, they are at least comparative.

It will be observed that the average value of the specific surface of the fly ashes of low carbon content was 2750 sq. cm. per gram. The calculated specific surface of an average portland cement is perhaps 1400 sq. cm. per gram. This difference in specific surface indicates that the fly ashes contained a much larger number of very fine particles than does the average portland cement. The results of tests have indicated that in portland cement there are few, if any, particles of diameter less than 2 microns. For some of the fly ashes it appears that there may be a large number of particles of diameter less than 1 micron.

Activity of Fly Ashes

Of the several methods which have been employed in attempting to evaluate the puzzolanic activity (ability to combine with lime) of fly ashes, the one showing the highest degree of correlation with concrete strengths at the later ages consisted in determining the compressive strength of mortars composed of a mixture of 2 parts of fly ash, 1 part of hydrated lime (high-calcium finishing lime), and 9 parts of standard Ottawa sand. The test pieces, which were 2 by 4-in. cylinders, were cured in sealed containers at a temperature of 70° F. for the first 12 hr., 100° F. for the second 12 hr., and 130° F. thereafter. They were tested in compression at ages of 7 and 28 days. The elevated temperatures are considered to accelerate the chemical reaction between fly ash and lime, so that the results of the tests up to the age of 28 days may be taken as indicative of the contribution of the fly ash to the strength of concrete at the later ages.

In Table 8 are shown for two groups of fly ashes—one of low carbon content and the other of high carbon content—the maximum, minimum, and average compressive strengths of the puzzolan-lime-sand mortar cylinders. (Detailed results of the activity tests are shown for individual fly ashes in Table 6.)

It will be observed that on the average the strengths of puzzolan-lime-sand mortar were considerably higher for the fly ashes of low carbon content than for those of high carbon content.

While it does not appear that the results of the activity tests on a particular fly ash may be utilized in quantitatively determining the contribution of that fly ash to the strength of concrete, there appears to be a general relationship between the results of the activity test

*"A Hydrometer Method for Determining the Fineness of Portland-puzzolan Cement," by S. B. Biddle Jr. and Alexander Klein, A. S. T. M., 1936, p. 310.

TABLE 6—CHEMICAL AND PHYSICAL PROPERTIES OF PUZZOLANS

(Table continued opposite page)

Puzzolan[1]	Chemical Analysis, Per Cent											
	SiO_2	FeO[4]	Fe_2O_3	Al_2O_3[5]	TiO_2	CaO	MgO	SO_3[7]	H_2O (240° F.)	CO_2	C	Na_2O
1	2	3	4	5	6	7	8	9	10	11	12	13
Chicago fly ash	44.18	4.23	17.45	16.44	0.83	7.14	0.92	2.34	0.32	0.16	1.13	2.17
Cleveland fly ash	44.10	4.27	18.99	20.85	1.11	4.00	0.79	1.37	0.36	0.01	1.13	0.58
Indiana fly ash	41.70	4.23	11.77	27.88	1.21	3.44	0.89	0.92	0.27	0.07	3.27	0.67
West Penn fly ash	49.00	2.07	3.86	27.53	1.45	5.36	0.86	0.42	0.09	0.17	6.36	0.73
Union Elec. fly ash	47.00	2.81	9.99	19.94	1.01	5.02	0.92	1.76	0.41	0.06	7.42	0.92
Detroit fly ash	46.40	2.20	6.29	27.49	1.51	2.48	1.02	0.77	0.35	0.02	7.54	1.93
Duquesne fly ash	47.82	2.16	6.98	25.57	1.19	2.48	0.79	0.46	0.14	0.16	9.26	0.66
Long Island fly ash	40.26	2.33	9.92	29.74	1.47	1.60	0.62	0.89	0.53	0.05	10.42	0.43
Potomac fly ash	35.24	2.85	7.74	22.87	1.27	10.59	1.82	1.50	0.15	0.32	11.94	1.03
N. Y. Edison fly ash	39.18	2.76	10.55	27.77	1.19	1.22	0.56	0.58	0.44	0.00	13.55	0.39
Cos Cob fly ash	40.40	1.28	3.91	30.65	1.04	1.78	1.17	0.97	0.13	0.01	13.89	1.03
Stamford fly ash	37.84	2.35	9.96	24.41	1.00	1.68	1.15	1.28	0.13	0.00	15.68	1.15
N. Y. Steam fly ash	34.68	4.06	14.30	21.52	0.97	2.80	0.55	1.42	0.57	0.19	16.56	0.47
Hell Gate fly ash[3]	32.84	3.63	11.75	25.42	0.84	1.00	0.57	0.52	0.18	0.08	19.26	1.26
Boston fly ash No. 1	30.32	1.58	7.00	18.54	0.90	8.32	1.91	1.27	0.08	0.13	26.36	0.90
Boston fly ash No. 2	28.10	1.62	6.96	17.84	0.90	7.28	1.73	1.46	0.20	0.15	30.25	0.74
Average for 16 fly ashes	39.94	2.78	9.84	24.03	1.12	4.14	1.02	1.12	0.27	0.10	12.13	0.94
Pumicite[2]	72.34	——	1.35	13.27	(6)	0.66	0.38	Tr.	——	——	——	1.65
Limestone dust[3]	6.90	——	1.06	2.22	(6)	48.54	2.16	Tr.	——	——	——	——

[1]All puzzolans tested as received, except as noted.

[2]Ground.

[3]Oven-dried before test; originally contained 43.8 per cent moisture.

[4]Total Fe and ferrous compounds as FeO.

[5]Including all of ammonia group except iron and titanium.

[6]Included with Al_2O_3.

and the strength of concrete, as illustrated by the foregoing tabulation in which are given the results of compression tests on standard-cured concretes. As between the two groups of fly ashes, high strength of concrete is paralleled by high strength of puzzolan-lime-sand mortar; and low strength of concrete is paralleled by low strength of puzzolan-lime-sand mortar.

TABLE 7—FINENESS OF FLY ASHES, BY GROUPS

Group		% Carbon	Percentage Passing a No. 325 Sieve (43 Microns)	Percentage Finer Than 10 Microns	Specific Surface, sq. cm. per gram
3 Fly ashes of low carbon content	Range	1–3	90–91	30–40	2460–3220
	Avg.	2	90	33	2750
5 Fly ashes of high carbon content	Range	16–30	62–84	7–37	1510–2980
	Avg.	22	70	20	2150

TABLE 6

(Continued from opposite page)

Chemical Analysis, Per Cent				Sp. Gr.	Fineness				Activity		
					% Passing Sieve (Wet)		% Finer Than 10 Microns	Specific Surface, sq. cm. per gram[8]	Puzzolan-lime-sand Mortar[9]		
									W/C, by wt.	Compr. Str. p.s.i.	
K_2O	Ign. Loss	Insol. Res.	Water Soluble		No. 200 (74μ)	No. 325 (43μ)				7 da.	28 da.
14	15	16	17	18	19	20	21	22	23	24	25
1.82	1.50	71.53	6.72	2.54	95.6	90.5	40.0	3220	0.49	1120	1280
1.53	1.55	78.54	4.22	2.57	94.9	90.0	29.8	2460	0.51	1110	1390
2.56	3.91	77.87	3.12	2.46	95.8	90.8	30.0	2580	0.51	870	1160
2.09	6.56	85.54	2.56	2.29	95.5	89.3	36.0	3080	0.53	810	1350
1.89	7.89	77.10	5.44	2.27	74.2	67.4	30.3	2660	0.53	1110	1660
2.08	8.33	87.65	2.10	2.25	90.7	85.1	35.0	3060	0.56	1000	1220
1.85	9.82	88.51	2.12	2.21	72.0	64.5	27.2	2580	0.58	840	1270
1.11	11.87	86.24	1.66	2.38	94.7	90.9	50.2	3800	0.61	1050	1960
1.12	13.16	67.25	5.46	2.34	84.9	80.3	21.5	2390	0.55	600	730
1.14	14.66	85.75	1.22	2.33	86.8	79.6	34.6	2460	0.66	900	1310
1.82	15.30	89.21	1.43	2.18	86.0	78.0	35.0	3150	0.63	750	900
1.97	17.05	83.15	2.09	2.29	90.5	83.6	36.8	2980	0.59	1080	1290
1.20	18.12	81.28	3.12	2.33	72.8	63.5	28.0	2580	0.59	670	1010
0.72	20.98	81.53	2.70	2.27	85.9	74.0	18.0	1970	0.63	400	560
1.29	27.89	75.85	4.69	2.20	73.3	62.2	7.0	1510	0.65	390	730
1.24	31.56	76.14	4.75	2.21	78.3	68.8	11.0	1720	0.68	410	560
1.59	13.13	80.82	3.34	2.32	85.7	78.7	29.4	2640	0.58	820	1150
5.40	4.13	97.94	——	2.37	99.5	98.6	43.3	3280	0.54	780	810
——	38.14	——	——	2.73	95.5	12.3	48.0	4240	0.50	0	0

[7]Total sulfur and compounds as SO_3.

[8]By hydrometer; computed from area under particle-size distribution curve extended to 1 micron or to limiting size if above 1 micron.

[9]2 Parts of puzzolan, 1 part of hydrated lime, 9 parts of standard Ottawa sand, by weight; puzzolans oven-dried before test; 2 by 4-in. cylinders; curing at 70° F. for 12 hr., then at 100° F. for 12 hr., and then at 130°F.

PROPERTIES OF CEMENTS AND CONCRETES

Physical Properties of Cements

Results of the standard physical tests on cements are shown in Tables 3, 9, and 10. All of the cements were sound, as indicated by

TABLE 8—ACTIVITY OF FLY ASHES, BY GROUPS

Group		Per Cent Carbon	Specific Surface, sq. cm. per gram	Activity (Compr. Str. of Puzzolan-Lime-Sand Mortar), p.s.i.		Compr. Str. of 1:5.6 Concrete Containing Portland-Puzzolan Cement, p.s.i. (80 per cent P. C., 20 per cent Puzzolan		
				7 da.	28 da.	28 da.	3 mo.	1 yr.
3 Fly ashes of low carbon content	Range	1–3	2460–3220	870–1120	1160–1390	——	——	——
	Avg.	2	2750	1030	1280	4900	6090	7190
5 Fly ashes of high carbon content	Range	16–30	1510–2980	390–1080	560–1290	——	——	——
	Avg.	22	2150	590	830	4030	5080	5870

TABLE 9—RESULTS OF TESTS ON HIGH-EARLY-STRENGTH CEMENTS

(Table continued opposite page)

Clinker Composition, %		Puzzolan			Cement			Neat Paste[2]		
		Type	% Carbon	Specific Surface, sq. cm. per gram[1]	% of Puzzolan	% Pass. No. 325 Sieve (Wet)	Specific Surface, sq. cm. per gram	Norm. Cons., % water by wt.	Time of Setting, hr. : min. (Gillmore)	
3CS	3CA								Init.	Final
1	2	3	4	5	6	7	8	9	10	11
50	8	None	—	——	0	97.5	2350	24.5	1:00	1:45
50	8	Chicago fly ash[3]	1.1	3220	10	97.9	2570	25.0	1:15	2:05
					20	97.7	2430	24.8	1:20	2:20
					30	99.0	2350	25.8	1:40	2:35
					40	99.3	2400	26.5	2:05	3:20
56	8	None	—	——	0	98.0	2420	25.5	1:15	2:15
56	8	Indiana fly ash[3]	3.3	2580	10	98.6	2570	25.7	1:35	3:15
					20	99.1	2620	26.5	2:30	4:15
					30	99.4	2740	26.6	3:45	5:15
					40	99.7	2760	28.0	4:00	6:00
56	8	Pumicite[3]	—	3280	20	98.4	3090	27.0	2:00	3:15
					30	99.0	3210	29.0	3:00	4:30

the standard steam test on a neat-cement pat. All of the fly-ash cements set more slowly than the corresponding portland cements. Of the fly-ash cements containing 20 percent of fly ash, all except six which were high in carbon, low in fineness, or both, met the

TABLE 10—RESULTS OF TESTS ON ADMIXED PORTLAND-PUZZO

Clinker Composition of Portland Cement	Puzzolan[2]		Cement		Neat Paste[3]			Standard Mortar			
	Type	Spec. Surf., sq. cm. per gram	% of Puzzolan	Spec. Surf., sq. cm. per gram	Norm. Cons., % water by wt.	Time of Setting, hr.:min. (Gillmore)		Water-Cement Ratio, by wt.	Tensile Strength, p.s.i. (Briquets)		
						Init.	Final		1 da.	7 da.	28 da.
1	2	3	4	5	6	7	8	9	10	11	12
58% 3CS, 6% 3CA[1]	None	——	0	1590	22.5	2:20	3:30	0.41	170	340	420
	Chicago fly ash	3220	30	2080	21.8	3:40	6:20	0.40	135	315	395
		3220	50	2400	22.0	3:40	7:30	0.41	60	325	335
	Cleveland fly ash	2460	30	1850	23.8	3:40	5:45	0.42	120	300	365
		2460	50	2030	24.2	4:20	7:40	0.42	55	235	330
	Pumicite	3280	30	2100	28.0	4:00	6:10	0.45	115	285	390-
		3280	50	2440	31.0	3:10	6:20	0.47	70	205	355

A.S.T.M. requirement for 28-day strength of standard mortar containing portland cement. A few fly-ash cements containing 30 or 50 percent of fly ash also failed to meet this requirement. On the average, at the age of 28 days the tensile strength of standard-cured mortar for the fly-ash cements containing up to 20 percent of fly ash was about the same as that for the corresponding portland cement,

TABLE 9

(Continued from opposite page)

Water-Cement Ratio, by wt.	Standard Mortar: Tensile Strength, p.s.i. (Briquets)[5]				Water-Cement Ratio, by wt.	Concrete[4]: Compressive Strength, p.s.i. (3 by 6-in. Cylinders)[5]			
	1 da.	3 da.	7 da.	28 da.		1 da.	3 da.	7 da.	28 da.
12	13	14	15	16	17	18	19	20	21
0.42	300	380	435	460	0.43	2570	3540	4740	5720
0.43	320	385	440	475	0.43	2330	3560	4720	5870
0.43	305	390	420	500	0.43	2490	3560	4590	5950
0.43	315	390	425	515	0.43	2130	2990	4040	5600
0.44	260	340	425	510	0.43	1680	2850	3560	5260
0.43	300	380	400	440	0.46	2680	4170	4470	5890
0.43	260	345	370	450	0.46	2720	4030	4410	6470
0.44	250	340	375	470	0.45	2380	4170	4190	6200
0.44	200	310	315	415	0.45	2040	3580	3800	5780
0.45	170	270	290	330	0.45	1600	3000	3300	5230
0.44	250	325	370	455	0.46	2520	3920	4050	6420
0.45	250	320	340	460	0.48	1890	3330	3700	5700

[1]Before blending.
[2]Puzzolan added to corresponding portland cement of specific surface 1600 sq. cm. per gram and mixture ground for same time as that required to increase specific surface of portland cement from 1600 sq. cm. per gram to specific surface shown (approximately 2400 sq. cm. per gram).
[3]All cements sound in standard steam test on neat-cement pat.
[4]1 Part of cement by weight to 5.6 parts of 0 to ¾-in. local gravel; flow 60 per cent.
[5]Standard curing.

LAN CEMENTS CONTAINING HIGH PERCENTAGES OF PUZZOLAN

(Continued from opposite page)

Water-Cement Ratio, by wt.	1:5.6 Concrete[4]: Compressive Strength, p.s.i. (3 by 6-in. Cylinders)					Length Change, millionths (1½ x 1½ × 12-in. Bars)[6]				Neat Paste[7]: Heat of Hydration After 15 Min., calories per gram				Strength-Heat Ratio, p.s.i. per calories per gram[8]	
	Standard Curing			70-100°F. Curing[5]		Exp. in Fog	Net Contraction (Air Storage)								
	7 da.	28 da.	3 mo.	28 da.	3 mo.	28 da.	35 da.	56 da.	4 mo.	1 da.	3 da.	7 da.	28 da.	28 da.	3 mo.
13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28
0.44	3310	4560	5440	4610	5270	20	265	415	515	32	49	65	80	57	68
0.41	3050	4420	6030	5660	6710	20	175	305	410	25	48	59	71	62	85
0.39	2400	3870	5350	5200	5790	20	185	320	435	16	39	49	62	62	86
0.41	2890	4410	5880	5700	6550	15	240	390	495	26	43	54	70	63	84
0.42	2120	3410	4930	5110	5640	30	205	340	425	17	37	45	58	59	85
0.46	1610	3550	4950	5020	5970	25	355	560	665	25	39	53	71	50	70
0.48	1370	2810	4100	4400	5130	30	365	625	725	19	32	43	59	48	70

[1]Commercially ground.
[2]Chicago fly ash and Cleveland fly ash as received contained 1.1 per cent of carbon.
[3]A. S. T. M. standard test; all cements sound in standard test on neat-cement pat.
[4]1 Part of cement by weight to 5.6 parts of 0 to ¾-in. local gravel; flow 60 per cent.
[5]In sealed metal cans; 1 da. at 70° F., then at 100° F. until $\left\{\begin{matrix}4\text{ da.}\\8\text{ da.}\end{matrix}\right\}$ prior to test for $\left\{\begin{matrix}28\text{-da.}\\3\text{-mo.}\end{matrix}\right\}$ tests, then cooled slowly to 70°F.
[6]Specimens standard-cured in fog to 28 da., then stored at 70°F. in air af 50% R. H.; all length changes referred to length at 3 days.
[7]Water-cement ratio 0.40 by wt.; Carlson vane calorimeter; control temperature 70°F.
[8]Ratio of compressive strength of standard-cured concrete at age indicated to heat of hydration of neat-cement paste at 28 days.

NOTE: All portland-puzzolan cements blended by admixing for 10 min. in laboratory batch mill equipped with wood vanes.

and at later ages it was greater than that for the corresponding portland cement. Further, at the age of 1 year the tensile strength of briquets which were stored in air after 28 days was higher for the fly-ash cements tested than for the portland cement (Table 4, column 12).

Water Requirement to Produce Concrete of Given Consistency

It is desirable that the amount of water required to produce concrete of given consistency be low. In the case of many cements containing natural puzzolans, however, it is observed that their water requirement is higher than that of a corresponding portland cement.

In these investigations, the water requirement of concrete was determined by flow-table tests. In Table 11 are given typical results showing the effect of carbon content and fineness of fly ash. (For individual values of water-cement ratio, see Tables 3, 4, 9 and 10.)

TABLE 11—WATER-CEMENT RATIO REQUIRED TO PRODUCE 60 PER CENT FLOW OF 1:5.6 CONCRETE CONTAINING 0 TO ¾-IN. AGGREGATE

General-purpose portland cement; fly-ash cements contain 20 per cent of fly ash.

Fly Ash in Cement	Per Cent Carbon in Fly Ash	Specific Surface of Fly Ash, sq. cm. per gram	Water-Cement Ratio of Concrete, by wt.
None	—	—	0.43
Chicago	1	3220	0.40
Cleveland	1	2460	0.42
West Penn	6	3080	0.43
Long Island	10	3800	0.46
Avg. for 3 fly ashes low in carbon	2	2750	0.41
Avg. for 5 fly ashes high in carbon	22	2150	0.49
Avg. for 6 fine fly ashes	9	3220	0.45
Avg. for 3 coarse fly ashes	25	1730	0.50

It will be noted that for the cement containing a fly ash which was low in carbon and of moderately high fineness (Chicago fly ash), the water requirement was less than that for the corresponding portland cement. The cement containing a fly ash which was moderately low in carbon and of moderately high fineness (West Penn fly ash) required the same amount of water as the portland cement. The cement containing a fly ash which was moderately high in carbon and of high fineness (Long Island fly ash) exhibited a high water requirement. Further, the water requirement was less for the average of 3 fly ashes low in carbon than for the average of 5 fly ashes high in carbon.

The effect of fineness of fly ash upon water requirement of concrete is shown in Table 11. It is seen that the water requirement was less for the average of 6 fine fly ashes than for the average of 3 coarse fly ashes. However, the comparison is not direct, as the two groups of fly ashes had different carbon contents. A direct comparison is obtained between Chicago fly ash, which was fine, and

Cleveland fly ash, which was fairly coarse. Each of these fly ashes contained 1 percent of carbon. In the tabulation, it is seen that the cement containing the finer fly ash exhibited the lower water requirement.

In general (as shown in Tables 3, 4, 9 and 10), for those fly-ash cements which required about the same amount of water as the corresponding portland cement, the water requirement was not appreciably affected by variation in the amount of fly ash. For those fly-ash cements which required more water than the portland cement, in general the larger the percentage of fly ash the greater the water requirement. For those fly-ash cements which required less water than the portland cement, the larger the percentage of fly ash the less the water requirement.

The foregoing comparisons point to the desirability of a fly ash of reasonably low carbon content and reasonably high fineness, and indicate that for a cement containing such a fly ash the water requirement to produce a concrete of a given consistency may be less than that for a portland cement. In this regard, such fly ashes differ from most other puzzolans, natural or artificial, since ordinarily a portland-puzzolan cement is recognized as requiring as much or more water than the corresponding portland cement. The obvious advantages which may accrue from the use of a fly ash of low water requirement as a puzzolanic replacement for portland cement are, first, those related to improved strength, watertightness, and volume constancy due to lowered water-cement ratio, and second, those related to improved workability on the basis of equal water-cement ratios.

Compressive Strength of Concrete

The effect of a moderate percentage—20 percent—of fly ash upon compressive strength of concrete is indicated by Table 12, in which are shown the compressive strengths of standard-cured concretes of the same mix and same consistency at various ages from 7 days to 1 year. The concrete contained 6.4 sacks of cement (94 lb. per sack) per cu. yd., and the maximum size of aggregate was ¾ in.; it might be compared to the mix normally employed in building construction. The same portland cement was used throughout, and the fly ash was mixed with the portland cement without grinding, so that the portland-cement constituent of the fly-ash cement was of the same fineness as the portland cement used without admixture.

Although at the ages of 7 and 28 days the compressive strength of concrete was higher for the portland cement than for any of the fly-ash cements, at the ages of 3 months and 1 year the concrete strength

TABLE 12—COMPRESSIVE STRENGTH OF STANDARD-CURED 1:5.6 CONCRETE CONTAINING FLY ASHES OF DIFFERENT CARBON CONTENT AND DIFFERENT FINENESS

General-purpose portland cement; portland-puzzolan cements contain 20 per cent of puzzolan.

Puzzolan in Cement	Per Cent Carbon in Fly Ash	Specific Surface, sq. cm./g.		W/C of Concrete, by wt.	Compressive Strength of Concrete, p.s.i.			
		Puzzolan	Cement		7 da.	28 da.	3 mo.	1 yr.
None	—	—	1600	0.43	4040	5310	5790	6810
Chicago fly ash	1	3220	1920	0.40	3760	5190	6230	7610
Cleveland fly ash	1	2460	1770	0.42	3270	4510	5700	6760
West Penn fly ash	6	3080	1900	0.43	3370	4910	6260	7450
Long Island fly ash	10	3800	2040	0.46	3220	4910	6390	7080
Avg. for 3 fly ashes low in carbon	2	2750	1830	0.41	3520	4900	6090	7190
Avg. for 5 fly ashes high in carbon	22	2150	1710	0.49	2770	4030	5080	5870
Avg. for 6 fine fly ashes	9	3220	1920	0.45	3220	4610	5940	6990
Avg. for 3 coarse fly ashes	25	1730	1630	0.50	2690	3890	4870	5630
Pumicite	—	3280	1940	0.44	3180	4590	5200	6160

of the cements containing fly ash of low carbon content and high fineness was higher than that for the portland cement.

Between the ages of 7 days and 3 months, the gain in strength of the concretes containing the four selected fly-ash cements averaged about 2700 p.s.i., as against approximately 1800 p.s.i. for the portland cement and 2000 p.s.i. for the portland-pumicite cement. Between the ages of 28 days and 1 year, the gain in strength of the concretes containing the four fly-ash cements averaged about 2300 p.s.i., whereas the gain in strength was only 1500 p.s.i. for the portland cement and 1600 p.s.i. for the portland-pumicite cement.

Effect of Carbon Content and Fineness of Fly Ash.—The effect of carbon content of fly ash upon compressive strength of concrete is shown in Table 12, for two groups of fly ashes of different carbon content. It is seen that the group of cements containing fly ashes of low average carbon content (2 percent) exhibited considerably higher concrete strengths at all ages than did the group of cements containing fly ashes of high average carbon content (22 percent).

With regard to the groups of cements containing fly ashes of different degrees of fineness, the group of cements containing fine fly ashes (average specific surface 3220 sq. cm. per gram) exhibited considerably higher concrete strengths at all ages than did the group of cements containing coarse fly ashes (average specific surface 1730 sq. cm. per gram). However, the carbon content of the two groups of fly ashes differed widely, so that the comparison is not direct. A direct comparison is obtained between Chicago fly ash, which was moderately fine, and Cleveland fly ash, which was moderately coarse, since both of these fly ashes had the same carbon content. The cement con-

taining the finer fly ash exhibited considerably higher concrete strength at all ages from 7 days to 1 year.

These results make it appear that a moderate amount of fly ash of reasonably low carbon content and reasonably high fineness, while contributing somewhat less to the early strength of concrete than would a corresponding weight of portland cement, contribute considerably more to the ultimate strength. The high strength of concretes containing fly-ash cement stamps the fly ashes of satisfactory composition and fineness as unique among the puzzolans.

Effect of Chemical Composition and Fineness of Portland Cement.— To investigate the effect of chemical composition of the portland-cement constituent upon certain properties of concrete, Chicago fly ash was admixed with each of four portland cements each of different chemical composition. The tricalcium-silicate (3CS) and tricalcium-aluminate (3CA) contents of these four portland cements are given in Table 13, together with the results of the compression tests of concrete. It is seen that as regards concrete strength, for the compositions and finenesses employed in this investigation the greatest benefit in substituting the fly ash for portland cement was in general obtained when the portland-cement constituent was moderately high or high in lime.

TABLE 13—COMPRESSIVE STRENGTH OF STANDARD-CURED 1:5.6 CONCRETE CONTAINING PORTLAND CEMENTS OF DIFFERENT COMPOSITION AND FINENESS ADMIXED WITH 20 PER CENT OF CHICAGO FLY ASH

Portland Cement				Compressive Strength of Concrete								
				p.s.i.						Ratio, % (Fly-Ash to Portland)		
				Portland Cement			Fly-Ash Cement					
Type	% 3CS	% 3CA	Spec. Surf., sq. cm./g.	28 da.	3 mo.	1 yr.	28 da.	3 mo.	1 yr.	28 da.	3 mo.	1 yr.
High-early-strength	60	8	2220	6030	6340	6920	6010	6880	7860	100	108	114
Normal	53	12	1590	5460	6100	6620	4890	6450	7920	90	106	120
General-purpose	50	6	1600	5310	5790	6810	5190	6230	7610	98	108	112
Low-heat	42	4	1600	4880	5770	6960	4020	5750	7170	82	100	103
General-purpose	50	6	2220	5750	6390	7260	5120	6170	7420	89	97	102
			1600	5310	5790	6810	5190	6230	7610	98	108	112
			1170	4070	4520	5710	3450	4770	6300	85	105	110

For three fly-ash cements each containing 20 percent of Chicago fly ash, the influence of fineness of the portland-cement constituent upon compressive strength of concrete is also indicated by the values given in Table 13. It will be observed that on the whole the greatest benefit to concrete strength in substituting the fly ash for portland cement was obtained in the case of the portland cement of normal fineness, *i.e.*, 1600 sq. cm. per gram.

High-Early-Strength Portland-Puzzolan Cements.—In Table 9 are given the results of a series of tests to investigate the possibilities of producing fly-ash cements having high-early-strength characteristics. The two portland cements employed in these tests were of a fineness represented by a nominal specific surface of 2400 sq. cm. per gram. The puzzolans—Chicago fly ash, Indiana fly ash, and pumicite—were the same as those used in the tests previously described. The fly ashes were employed as replacements in percentages of 10, 20, 30, and 40. The materials were blended by intergrinding, as follows: The portland cement was ground to a specific surface of 1600 sq. cm. per gram, then the puzzolan was added and the mixture was ground for the same length of time as that required to increase the specific surface of the portland cement from 1600 to 2400 sq. cm. per gram. This procedure resulted in specific surfaces of portland-puzzolan cement which were equal to, or higher than, that of the corresponding portland cement.

When the Chicago fly ash (low carbon, moderately high fineness) was used as a replacement for portland cement in amounts of 10 or 20 percent, at ages of 1 to 28 days the compressive strengths of concrete were substantially the same for the fly-ash cements as for the corresponding portland cements. When 30 percent of this fly ash was used as a replacement, at the early ages the concrete strengths were relatively low for the fly-ash cements but at the age of 28 days the concrete strengths were substantially the same for the fly-ash cements as for the portland cements.

When the Indiana fly ash (low carbon, moderately low fineness) was used as a replacement for 10 percent of the portland cement, at ages of 1 to 28 days the compressive strength of concrete for the fly-ash cement was equal to or greater than that for the portland cement. When 20 or 30 percent of this fly ash was used as a replacement, at the early ages the concrete strengths were relatively low for the fly-ash cements but at the age of 28 days the concrete strengths for the fly-ash cements were approximately equal to those for the portland cements. Similarly, the portland-pumicite cements exhibited relatively low strengths at early ages but relatively high strengths at the age of 28 days.

For the particular combinations of materials here considered, the tests indicate that replacement of these types of portland cement with fly ash up to perhaps 20 percent do not impair the compressive strength of concrete at early ages; and that the strength at ages as early as 28 days may be improved by replacements of fly ash up to perhaps 30 percent. It might be expected that, under satisfactory conditions of

curing, the high-early-strength fly-ash cements would exhibit substantially higher concrete strengths at the later ages than would the corresponding high-early-strength portland cements.

Effect of Intergrinding.—Certain of the fly ashes were interground as well as admixed with the portland cement. Each mixture of clinker and puzzolan was ground under the same mill conditions and for the same period as that required to grind the portland cement to a specific surface of 1600 sq. cm. per gram. The effect of intergrinding is shown in Table 14.

TABLE 14—COMPRESSIVE STRENGTH OF STANDARD-CURED 1:5.6 CONCRETE CONTAINING INTERGROUND AND ADMIXED FLY-ASH CEMENTS

General-purpose portland cement.

Fly Ash			Per Cent of Fly Ash	Specific Surface, of Cement, sq. cm/g.		Compressive Strength of Concrete					
						7 da.			1 yr.		
						p.s.i.		Ratio, Per Cent	p.s.i.		Ratio, Per Cent
Type	Per Cent Carbon	Spec. Surf., sq. cm. /g.		Adm.	Intg.	Adm.	Intg.		Adm.	Intg.	
Chicago	1	3220	20	1920	2110	3760	4090	109	7610	7940	104
Long Island	10	3800	20	2040	2790	3220	3390	105	7080	6480	92
Potomac	12	2390	20	1760	2320	3270	3600	110	6840	6970	102
Chicago	1	3220	30	2090	2570	3210	3150	98	7770	6930	89

In all cases the effect of intergrinding was to increase the fineness of cement. At the earliest age of test—7 days—the interground cements containing 20 percent of fly ash exhibited concrete strengths which were on the average 8 percent higher than the average strength for the corresponding admixed cements. At the age of 1 year, the average strengths for the interground cements containing 20 percent of fly ash were about equal to those for the admixed cements. For the interground cement containing 30 percent of Chicago fly ash (low carbon, moderately high fineness), the concrete strength at the age of 7 days was about the same as that for the corresponding admixed cement, and at the age of 1 year was lower than that for the admixed cement. For these materials and conditions of grinding, it appears that intergrinding is of benefit only at early ages.

Effect of Richness of Mix.—The effect of richness of mix upon the compressive strength of standard-cured concrete containing fly-ash cement is indicated in Table 15. The maximum size of aggregate was ¾ in., and the concrete was of fixed consistency (flow 60 percent). The specimens were 3 by 6-in. concrete cylinders.

It will be seen that regardless of the richness of the mix the compressive strengths were higher for the concretes containing the fly-ash cements than for the concretes containing the portland cement. As between concretes of different cement content, the percentages of in-

TABLE 15—COMPRESSIVE STRENGTH OF CONCRETES CONTAINING VARIOUS AMOUNTS OF CEMENT

General-purpose portland cement.

Fly Ash			Per Cent of Fly Ash	Compressive Strength of Concrete at 3 Mo.					
				p.s.i.			Per Cent of Str. for P.C.		
Type	Per Cent Carbon	Spec. Surf., sq. cm. /g.		Rich 7.6 sks./c.y.	Normal 6.4 sks./c.y.	Lean 5.5 sks./c.y.	Rich 7.6 sks./c.y.	Normal 6.4 sks./c.y.	Lean 5.5 sks./c.y.
None.....	—	—	0	6500	5790	5410	100	100	100
Chicago.....	1	3220	20	6770	6230	5550	104	108	103
West Penn...	6	3080	20	7320	6260	5640	113	108	104
Long Island..	10	3800	20	6770	6390	5570	104	110	103

crease in strength due to the use of fly ash did not differ greatly; it thus appears that the relations discussed herein for concrete of normal cement content would likewise apply to concretes of high and low cement content.

Effect of Percentage of Fly Ash.—The effect of the amount of fly ash in a cement upon the compressive strength of concrete is summarized in Table 16 for the combinations of fly ash and portland cement in various percentages. (The data are taken from Tables 3 and 10.) For the fly-ash cements, each value in the body of the tabulation represents the ratio, in percent, of the compressive strength of concrete containing the fly-ash cement to the compressive strength of concrete containing the corresponding portland cement.

At the early ages of 7 and 28 days, the larger the percentage of fly ash the lower the strength of concrete. At the ages of 3 months and 1 year, however, for the cements containing Chicago fly ash (low in carbon, moderately high in fineness) the greater the amount of fly ash up to 30 percent the higher the strength. For both the Chicago fly ash and the Cleveland fly ash (low in carbon, moderately low in fineness) the concrete strengths were lower for a 50-percent replacement of portland cement by fly ash than for a 30-percent replacement. However, it is interesting to observe that, even with half of the portland cement replaced by fly ash, concrete strengths were obtained which at the age of 3 months were of the order of 5000 p.s.i.

In Table 16, it is seen that for the fly ash which was moderately low in carbon (West Penn), at the age of 1 year the concrete strength for a 30-percent replacement of portland cement by fly ash was about the same as that for a 20-percent replacement. For the fly ashes which were moderately high in carbon (Long Island and Potomac), the concrete strengths were lower for a 30-percent replacement than for a 20-percent replacement. Similar findings

TABLE 16—EFFECT OF PERCENTAGE OF FLY ASH UPON COMPRESSIVE STRENGTH OF STANDARD-CURED 1:5.6 CONCRETE

	Compound composition	50 Per Cent 3CS; 6 Per Cent 3CA				58 Per Cent 3CS; 6 Per Cent 3CA		
Portland Cement	Specific surface, sq. cm./g.	1600				1590		
	Age of test	7 da.	28 da.	3 mo.	1 yr.	7 da.	28 da.	3 mo.
	Compr. strength of concrete, p.s.i.	4040	5310	5790	6810	3310	4560	5440

	Fly Ash			Per Cent of Fly Ash	Ratio, in Per Cent, of Compressive Strength of Concrete Containing Fly-Ash Cement to Compressive Strength of Concrete Containing Corresponding Portland Cement						
	Type	Per Cent Carbon	Spec. Surf., sq. cm./g.								
Fly-Ash Cement	Chicago	1	3220	10	99	99	106	110	—	—	—
				20	93	98	108	112	—	—	—
				30	80	94	110	114	92	97	111
				50	—	—	—	—	73	85	98
	Cleveland	1	2460	30	—	—	—	—	87	97	108
				50	—	—	—	—	64	75	91
	West Penn	6	3080	20	83	92	108	109	—	—	—
				30	71	83	102	108	—	—	—
	Long Island	10	3800	20	80	92	110	104	—	—	—
				30	65	80	100	99	—	—	—
	Potomac	12	2390	20	81	90	101	100	—	—	—
				30	64	75	91	93	—	—	—

were shown in the high-early-strength tests discussed previously and reported in Table 9.

From the foregoing comparisons it may be concluded that in so far as compressive strength of concrete is concerned, the optimum percentage of a fly ash of low carbon content and moderately high fineness may be 30 or more; and that except for fly ashes which are of high carbon content or low fineness, or both, a 30-percent replacement will produce a concrete for which the compressive strength will within a few months equal or exceed that which will be produced by the corresponding portland cement.

A point of further interest is the fact that when a suitable fly ash was employed, replacements as high as 50 percent resulted in concrete strengths which were not unduly low, and even at the early ages the reduction in strength was considerably less than 50 percent. This fact is of importance in regard to the development of cements for mass-concrete construction.

Effect of Curing under Mass Conditions.—In order to determine the compressive strength of fly-ash concretes under conditions of hydration similar to those which obtain in mass-concrete structures, specimens in the form of 3 by 6-in. concrete cylinders were cured in sealed containers, at 70° F. for 1 day and thereafter at 100° F., and

at various ages were tested in compression. The results of the tests are given in Table 17. For the fly-ash cements, each value represents the ratio, in percent, of the compressive strength of the concrete containing the fly-ash cement to the strength of the concrete containing the corresponding portland cement.

TABLE 17—COMPRESSIVE STRENGTH OF 1:5.6 CONCRETE CURED UNDER STANDARD AND MASS CONDITIONS

Portland Cement			Fly Ash			Per Cent of Fly Ash	Compressive Strength of Concrete, p.s.i.[1]					
							Standard Curing			70-100° F. Curing		
Per Cent 3CS	Per Cent 3CA	Spec. Surf., sq. cm. /g.	Type	Per Cent Carbon	Spec. Surf., sq. cm. /g.		7 da.	28 da.	3 mo.	7 da.	28 da.	3 mo.
50	6	1600	None	—	——	0	4040	5310	5790	4300	5320	5790
			Chicago	1	3220	20	93	98	108	98	114	117
			West Penn	6	3080	20	83	92	108	90	113	110
			Long Island	10	3800	20	80	92	110	86	106	106
			Potomac	12	2390	20	81	90	101	88	105	102
			Hell Gate	19	1970	20	70	73	85	75	88	89
58	6	1590	None	—	——	0	3310	4560	5440	—	4610	5270
			Chicago	1	3220	30	92	97	111	—	123	127
						50	73	85	98	—	113	110
			Cleveland	1	2460	30	87	97	108	—	124	124
						50	84	75	91	—	111	107

[1]For the fly-ash cements, values are percentages of the compressive strength exhibited by the corresponding portland cement.

A study of the values given in Table 17 shows that the conditions of mass curing are particularly favorable to the development of high strengths of fly-ash-cement concretes as compared with the strengths of corresponding portland-cement concretes.

It will be observed that under mass curing conditions, even at the relatively early age of 28 days, the strengths of the concretes containing fly ashes of low carbon content were considerably greater than those of the concretes containing corresponding portland cement without admixture, even for fly-ash replacements as high as 50 percent. The test results make it appear that, so far as strength is concerned, properly constituted fly-ash cements as a group are considerably superior to portland cements where the conditions of curing involve temperatures above normal. When (as shown later) it is considered that the larger replacements of fly ash result in a substantial reduction in the heat of hydration as compared with that which would be obtained with the corresponding portland cement, the possibilities of the use of high-replacement fly-ash cements for mass-concrete construction appear to be very attractive.

Elasticity of Concrete

For selected cements, the initial tangent modulus of elasticity of

TABLE 18—MODULUS OF ELASTICITY OF STANDARD-CURED 1:5.6 CONCRETE
General-purpose portland cement.

Fly Ash			Per Cent of Fly Ash	Initial Tangent Modulus of Elasticity, million p.s.i.			
Type	Per Cent Carbon	Specific Surface, sq. cm/g		7 da.	28 da.	3 mo.	1 yr.
None	—	—	0	5.4	5.9	5.7	6.2
Chicago	1	3220	20	5.8	6.0	6.1	6.8
West Penn	6	3080	20	5.2	5.9	6.0	6.4
Long Island	10	3800	20	4.9	5.7	6.1	6.4

standard-cured 1:5.6 concrete at ages of 7 days to 1 year is shown in Table 18.

In general, there was a close relation between the strength and the modulus of elasticity of the concretes, for the fly-ash cements as well as for the portland cements. On the average, at the early ages the modulus of elasticity was slightly lower for the fly-ash cements as a group than for the portland cements, and at the later ages it was somewhat higher; but the differences were not sufficiently great to become a factor in design.

Volume Changes of Concrete

In previous investigations it has been observed that concretes containing portland-puzzolan cements derived from natural puzzolans in general tend to shrink considerably more upon drying than do concretes of the same mix containing portland cement. This characteristic is considered undesirable, particularly in the case of thin concrete structures subjected to prolonged drying conditions, inasmuch as excessive shrinkage tends to produce cracks.

Since the drying shrinkage of concrete is related to the reduction in moisture content, which in turn is influenced by the area exposed to drying, by the size of the pore spaces within the concrete mass, and by the extent of hydration of the cement, it was felt that possibly the shrinkage of small concrete bars exposed on all sides to drying conditions might not be a true index to the shrinkage which would occur in ordinary concrete structures, where, as compared to the volume of the concrete mass, the areas exposed to drying are relatively small. In order to investigate the effect of the mass-area factor upon drying shrinkage, volume changes were determined for two types of concrete specimen—6-in. cubes and 1½ by 1½ by 12-in. bars.

All specimens were moist-cured up to the age of 28 days and thereafter were stored in air at 70° F. and 50 percent relative humidity. At the age of 28 days, five sides of each of the cubes were waterproofed so that all evaporation would take place from one face. Periodically measurements were taken to determine the change in length of concrete bars and the change in length of concrete cubes along planes

parallel with the exposed face and at distances therefrom of ¼ in., 2 in., and 5¾ in. The results of the shrinkage tests up to the age of 1 year are summarized in Table 19. (For further details see Tables 3 and 4.)

TABLE 19—CONTRACTION OF 1:5.6 CONCRETE BARS AND CUBES CONTAINING FLY ASHES OF DIFFERENT CARBON CONTENT AND DIFFERENT FINENESS

General-purpose portland cement; portland-puzzolan cements contain 20 per cent of puzzolan.

Puzzolan in Cement	Per Cent Carbon in Fly Ash	Specific Surface, sq. cm./g.		Contraction, millionths (Storage at 70°F. in Fog to 28 Da., Then in Dry Air)							
				1½ x 1½ × 12-In. Bars[1]		6-In. Cubes[2]					
						¼ In. from Exposed Face		2 In. from Exposed Face		5¾ In. from Exposed Face	
		Puzzolan	Cement	4 mo.	1 yr.	4 mo.	1 yr.	4 mo.	1 yr.	4 mo.	1 yr.
None	—	—	1600	440	475	410	515	160	290	5	95
Chicago fly ash	1	3220	1920	455	490	375	480	145	245	5	30
West Penn fly ash	6	3080	1900	505	530	445	570	180	310	10	45
Long Island fly ash	10	3800	2040	535	570	365	445	155	270	5	35
Avg. for 3 fly ashes low in carbon	2	2750	1830	490	520	—	—	—	—	—	—
Avg. for 5 fly ashes high in carbon	22	2150	1710	550	600	—	—	—	—	—	—
Avg. for 6 fine fly ashes	9	3220	1920	535	565	—	—	—	—	—	—
Avg. for 3 coarse fly ashes	25	1730	1620	550	600	—	—	—	—	—	—
Pumicite	—	3280	1940	575	620	—	—	—	—	—	—

[1]Net contraction, referred to the length at 3 days.
[2]Gross contraction, referred to the length at 28 days.

With regard to the results of tests on 6-in. cubes, it will be observed that at the age of 1 year, the shrinkage of the concretes containing fly-ash cements, at a distance of 5¾ in. from the exposed face, was on the average only about one third the shrinkage of the concrete containing the corresponding portland cement. Also, for the two finer fly ashes (Chicago and Long Island) the shrinkage of the cubes at distances ¼ and 2 in. from the exposed face was less than for the portland cement.

It is perhaps fair to assume that concretes containing properly constituted fly-ash cements possess a finer pore structure than do concretes containing normal portland cement, that due to this finer pore structure moisture is less readily withdrawn through evaporation from exposed surfaces, and that in ordinary structures containing fly-ash cement subjected to normal drying conditions the shrinkage may be no more, and perhaps may be less, than for corresponding structures containing portland cement.

With regard to the results of tests on 1½ by 1½ by 12-in. bars—which tests represent extreme conditions of drying, particularly when the relatively slow rate of hydration of puzzolanic cements is consid-

ered—it will be observed that at the age of 1 year the shrinkage was less for the portland cement than for any of the puzzolanic cements, although for the better fly ashes the differences were not great.

It will be observed that on the average, concretes containing fly ashes of low carbon content or high fineness shrank less than did corresponding concretes containing fly ashes of high carbon content or low fineness, and that the concrete containing pumicite (which among natural puzzolans exhibits a fairly low shrinkage) shrank more than did the corresponding concrete for any fly-ash cement.

A fact of further interest is that, as between the fly-ash cements, generally the higher the strength of concrete the less the contraction.

Effect of Percentage of Fly Ash.—The effect of the amount of fly ash in a portland-puzzolan cement on the contraction of concrete upon drying is shown in Table 20. The specimens were 1½ by 1½ by 12-in. bars. For the fly-ash cements, the values in the body of the table are ratios, in percent, of the contraction of concrete containing the fly-ash cement to the contraction of concrete containing the corresponding portland cement.

TABLE 20—EFFECT OF PERCENTAGE OF FLY ASH UPON CONTRACTION OF 1:5.6 CONCRETE BARS

Standard curing to 28 days, then storage at 70°F. in air of 50 per cent relative humidity. Contraction referred to length at 3 days.

Portland Cement	Compound composition			50% 3CS; 6% 3CA		58% 3CS; 6% 3CA
	Specific surface, sq. cm./g.			1600		1590
	Age of test			4 mo.	1 yr.	4 mo.
	Contraction of concrete, millionths			440	475	515

	Puzzolan			Per Cent of Puzzolan	Ratio in Per Cent, of Contraction for Portland-Puzzolan Cement to Contraction for Corresponding Portland Cement		
	Type	Per Cent Carbon	Specific Surface, sq. cm./g.				
Portland-Puzzolan Cement	Chicago fly ash	1	3220	10	109	108	—
				20	103	103	—
				30	104	104	80
				50	—	—	85
	Cleveland fly ash	1	2460	30	—	—	96
				50	—	—	83
	West Penn fly ash	6	3080	20	115	112	—
				30	111	108	—
	Long Island fly ash	10	3800	20	121	120	—
				30	121	119	—
	Potomac fly ash	12	2390	20	115	114	—
				30	133	133	—
	Pumicite	—	3280	30	—	—	129
				50	—	—	141

For the fly ashes of low carbon content, within the limits of this investigation the percentage of fly ash had in general no great effect upon the amount of contraction. For the fly ash of high carbon content and low fineness (Potomac fly ash), the contraction increased as the percentage of replacement was increased.

TABLE 21—CONTRACTION OF BARS OF 1:5.6 CONCRETE CONTAINING VARIOUS PORTLAND CEMENTS ADMIXED WITH 20 PER CENT OF CHICAGO FLY ASH

Portland Cement				Contraction, millionths (Storage at 70°F. in Fog to 28 Da., Then in Air to 1 Yr.)[1]		Ratio, % (Fly-Ash to Portland)
Type	Per Cent 3CS	Per Cent 3CA	Spec. Surf., sq. cm. per g.	Portland Cement	Fly-Ash Cement	
High-early-strength	60	8	2220	510	440	86
Normal	53	12	1590	495	505	102
General-purpose	50	6	1600	475	490	103
Low-heat	42	4	1600	530	490	92
General-purpose	50	6	2220	530	510	96
			1600	475	490	103
			1170	600	525	87

[1]Net contraction, referred to the length at 3 days.

Effect of Composition and Fineness of Portland Cement.—The shrinkage of 1½ by 1½ by 12-in. concrete bars containing several types of portland cement and the shrinkage of bars containing corresponding fly-ash cements are summarized in Table 21.

For the portland cements of normal fineness and normal composition as to tricalcium silicate, the replacement of fly ash had little influence upon the shrinkage of concrete bars. With the high-early-strength portland cement, a replacement of fly ash resulted in a substantial reduction in shrinkage of concrete. The same is true with the low-heat cement of normal fineness and the general-purpose cement of low fineness.

It will be observed that the least shrinkage occurred in the bars containing the high-early-strength fly-ash cement.

While the reasons for these differences in behavior are not altogether clear, it would seem reasonable to suppose that in the case of the high-early-strength cement, which is high in lime and of high fineness, conditions would be most favorable for an early combination between the fly ash and the lime liberated during the process of hydration of the portland-cement constituent; also, for portland cement of low fineness, the addition of fly ash would tend to reduce the size of the pore structure and hence to retard the rate of moisture loss through evaporation, insuring a more complete hydration of cement than would be possible if the moisture loss were rapid.

Durability of Concrete

For exposed structures the ability of concrete to resist the action of weather is important. Furthermore, certain concrete structures—notably harbor works and structures located in alkali ground—are subjected to the action of aggressive waters. It is known that among the factors which influence the durability of concrete under such conditions are the composition and fineness of cement.

Resistance to Freezing and Thawing.—To determine their relative resistance to the action of weather, concretes containing selected cements were subjected to an accelerated test which consisted in the alternate freezing and thawing of 3 by 6-in. cylindrical test specimens. The specimens were standard-cured for 5 months and then were subjected to thirty 24-hr. cycles of freezing and thawing while saturated. Each cycle comprised 16 hr. at 0° F. and 8 hr. at 70° F. The effect of the treatment was determined by visual inspection of the specimens and by their compressive strength as compared with the compressive strength of concretes of the same age which had not been subjected to the treatment. Results of the freezing-and-thawing test are given in Table 22. (The values are taken from Table 4.)

TABLE 22—RESISTANCE TO FREEZING AND THAWING OF 3 BY 6-IN. CYLINDERS OF 1:5.6 CONCRETE

General-purpose portland cement; fly-ash cements contain 20 per cent of fly ash.

Fly Ash			Concrete				
Type	Per Cent Carbon	Specific Surface, sq. cm./g.	Loss in Weight, per cent	Appearance Rating	Compr. Strength at 6 Mo.		
					p.s.i.		Ratio, Treated to Untreated
					Untreated	Treated	
None	—	—	0.2	9	6310	6050	0.96
Chicago	1	3220	0.2	9	6950	6780	0.98
West Penn	6	3080	0.1	9	6800	5060	0.75
Long Island	10	3800	0.0	9	6650	5490	0.83
Potomac	12	2390	0.1	9	6420	4920	0.77
Hell Gate	19	1970	0.0	9	5480	3560	0.65

On the basis of appearance of the specimens there was no difference between the fly-ash cements and the corresponding portland cements. The loss in weight during the freezing-and-thawing treatment on the average was slightly less for the group of fly-ash cements than for the portland cement. For all cements, the compressive strength of the treated specimens was lower than that of the untreated specimens. The fly ash which was lowest in carbon and of moderately high fineness (Chicago fly ash) exhibited a higher compressive strength of treated specimens and a higher strength ratio than did the portland cement. The cement exhibiting the greatest loss in strength due to the treatment contained the fly ash highest in carbon content and of lowest fineness (Hell Gate fly ash).

Resistance to Sodium Sulfate.—To determine their relative resistance to the action of aggressive waters, concretes containing selected cements were subjected to an accelerated test which consisted in immersing the test specimens in a 10-percent solution of sodium sulfate. The specimens, which were 3 by 6-in. cylinders of 1:5.6 concrete, were standard-cured for 28 days, then immersed in the solution for 5 months. The effect of the treatment was determined by visual inspection of the specimens and by comparing their compressive strength with that of corresponding specimens of the same age which had been continuously standard-cured. Results of the tests are given in Table 23. (The values are taken from Table 4.)

TABLE 23—RESISTANCE TO SODIUM SULFATE OF 3 BY 6-IN. CYLINDERS OF 1:5.6 CONCRETE

General-purpose portland cement.

Fly Ash				Concrete					
Type	Per Cent Carbon	Specific Surface, sq. cm. /g.	Per Cent of Fly Ash	Appearance Rating			Compressive Strength at 6 Mo.		
							p.s.i.		Ratio, Treated to Untreated
				4 mo.	5 mo.	6 mo.	Untreated	Treated	
None.....	—	——	0	10	9	7	6310	5560	0.88
Chicago.....	1	3220	20	10	10	8	6950	7120	1.03
West Penn...	6	3080	20	10	9	7	6800	6580	0.97
Long Island..	10	3800	20	10	10	8	6650	6400	0.96
Potomac.....	12	2390	20	9	8	5	6420	5830	0.91
Hell Gate....	19	1970	20	10	9	6	5480	5060	0.93

On the basis of appearance of specimens, the only two fly-ash cements which were not the equal of the corresponding portland cement were high in carbon and low in fineness as compared with the others. On the basis of the ratio of compressive strength of treated specimens to that of corresponding untreated specimens, all fly-ash cements were superior to the corresponding portland cement. It is significant that the Chicago fly ash exhibited a higher compressive strength for specimens which had been subjected to the sodium-sulfate treatment than for corresponding specimens which had been continuously standard-cured. Not only was the Chicago fly ash low in carbon and of moderately high fineness, but also among the fly ashes it was the lowest in alumina.

Heat of Hydration of Cement

In massive concrete structures it is desirable that the heat of hydration of the cement be low in order that the rise in temperature due to heat of hydration and the subsequent decline in temperature as the heat is gradually dissipated may be low. If this temperature change is large, stresses due to these thermal changes are likely to produce cracks. Even in walls having a thickness no greater than 2 or 3 feet,

where standard portland cements have been employed, in many cases cracks of considerable frequency and magnitude have been observed soon after the forms were removed.

The rate and amount of the heat of hydration of portland-puzzolan cement is influenced by the chemical composition and the fineness of the portland-cement constituent and by the character of the puzzolanic material. In the case of natural puzzolans, it has been observed that their contribution to this heat is substantially less than that for a corresponding amount of portland cement.

In Table 24 (values from Tables 4 and 10) there are given the results of heat-of-hydration tests on fly-ash cements for which the portland-cement constituent would fall in the moderately low-heat class, together with values for the corresponding portland cements and for the low-heat portland cement tested in this investigation.

TABLE 24—HEAT OF HYDRATION OF CEMENTS

Portland Cement			Fly Ash			Per Cent of Fly Ash	Heat of Hydration at 70°F., calories per gram (After 15 Min.)			
Per Cent 3CS	Per Cent 3CA	Specific Surface, cm²/g.	Type	Per Cent Carbon	Specific Surface, sq. cm./g.		1 da.	3 da.	7 da.	28 da.
50	6	1600	None	—	—	0	33	55	72	84
			Chicago	1	3220	20	29	51	62	75
			West Penn	6	3080	20	29	47	64	80
			Long Island	10	3800	20	29	50	67	79
			Potomac	12	2390	20	30	49	64	79
			Hell Gate	19	1970	20	32	51	64	76
58	6	1590	None	—	—	0	32	49	65	80
			Chicago	1	3220	30	25	48	59	71
						50	16	39	49	62
			Cleveland	1	2460	30	26	43	54	70
						50	17	37	45	58
42	4	1600	None (low-heat p. c.)			0	30	45	58	73

For all of the fly-ash cements, the heat of hydration at ages of 1 to 28 days was less than that for the corresponding portland cements. The cements containing 30 to 50 percent of fly ash were low-heat cements; and those containing 50 percent of fly ash generated less heat than any of the low-heat portland cements which have so far been employed in construction.

Plastic Flow of Mortar

The plastic flow of mortars containing selected cements was determined by subjecting 3 by 3 by 40-in. mortar beams to sustained load and by observing the deflections at intervals. The mortar contained 1 part of cement by weight to 3.25 parts of a 0 to No. 4 graded local sand; the water-cement ratio was that required to produce a flow of 40 percent. The specimens were standard-cured in fog throughout

the period of test, 3 months. The beams were loaded at the third-points on a span of 39 inches.

Beginning at the age of 7 days, half of the beams for each condition were subjected to total sustained load (including weight of beam and apparatus) as follows: From 7 to 14 da., 100 lb.; from 14 to 21 da., 150 lb.; from 21 to 28 da., 200 lb.; and from 28 da. to 3 mo., 250 lb. At the age of final test, 3 months, the load was released for 45 minutes, then load was applied at a normal testing speed until failure occurred. At the same age, the specimens which had not been subjected to sustained load were also loaded at a normal rate to failure. The center deflections of the beams were observed before and after the time of each change in sustained load and at intervals during the period of rapid loading. The plastic (total minus elastic) deflection between 7 and 28 days is shown in Table 25.

TABLE 25—PLASTIC DEFLECTION AT CENTER OF MORTAR BEAMS SUBJECTED TO SUSTAINED LOAD AT THE AGE OF 7 DAYS

Portland Cement			Fly Ash			Per Cent of Fly Ash	W/C of Mortar, by wt.	Plastic Deflection at Center of Beam per Pound of Total Load, millionths of inches (Between 7 and 28 Days)
Per Cent 3CS	Per Cent 3CA	Specific Surface, sq. cm. /g.	Type	Per Cent Carbon	Specific Surface, sq. cm. /g.			
60	8	2220	None	—	——	0	0.41	22
			Chicago	1	3220	20	0.40	25
50	6	1600	None	—	——	0	0.41	21
			Chicago	1	3220	20	0.40	24
			West Penn	6	3080	20	0.42	27
			Long Island	10	3800	20	0.44	24
			Potomac	12	2390	20	0.44	28

At these early ages, 7 to 28 days, the plastic deflection was consistently greater for the fly-ash cements than for the corresponding portland cements. At the later ages, however, the rate of hardening of the fly-ash cements exceeded that of the portland cements, so that the total flows to the final age of test (3 mo.) were not greatly different. This property of plastic flow at the early ages is considered desirable, in that differential stresses which may be set up in concrete during the early hardening period may be the more readily relieved.

Suggested Specification Requirements for Fly Ash

Throughout this paper, certain general comparisons have been drawn which make it appear that through the use of fly ashes of low carbon content and high fineness concretes can be produced which are at least equal, and in some respects superior, to concretes which can be produced with a corresponding portland cement.

In order that fly ash as an ingredient of concrete may be selected with the assurance that it will give satisfactory performance, it would

seem desirable that there be prepared a specification containing, in addition to a definition of the material, certain chemical and physical requirements.

For use in moderate percentages in a cement for general purposes, it is believed that a fly ash which meets the following requirements would produce a concrete the equal of that which would be produced if the portland cement were used alone.

1. Specific surface not less than 2500 sq. cm. per gram and not more than 4000 sq. cm. per gram, as determined by the hydrometer method with dispersion in water. Further the fineness should be such that not more than 12 per cent of the material is retained on the No. 325 sieve when the method of wet-sieving is employed.
2. Loss on ignition not to exceed 7 per cent. This will insure a fly ash of low carbon content. For all fly ashes there is very close agreement between ignition loss and carbon content.
3. Magnesium oxide content not to exceed 3 per cent.
4. Sulfur and its compounds not to exceed 3 per cent.
5. Silica not less than 40 per cent.
6. Activity to be such that the compressive strength of fly-ash-lime-sand-mortar, when prepared, cured, and tested in accordance with the method described herein, will be not less than 700 p.s.i. at 7 days and not less than 1100 p.s.i. at 28 days.
7. The physical character of the fly ash to be such that a thoroughly blended mixture containing 80 per cent of portland cement and 20 per cent of fly ash will require a percentage of water which does not exceed by more than 3 the percentage of water required to produce normal consistency for the same portland cement without fly ash.

Given proper powdered-fuel equipment and proper conditions of combustion, it would appear that a fly ash meeting these requirements might be readily produced in any steam plant from a rather wide variety of coal deposits.

CONCLUSIONS

The following general conclusions as regards the suitability of employing fly ash as a replacement for portland cement are based upon the results of tests up to the age of one year on fly ashes from 15 different sources, blended in percentages up to 50 with portland cement of seven compositions. All fly-ash replacements were made by weight, and in general the cements were blended by mixing, though in a few cases the cements were blended by intergrinding.

1. Though the fly ashes were derived from widely scattered coal deposits, the differences in chemical composition are not large, except as regards carbon content.

2. In general, fly ashes are much finer than are portland cements, some of the particles perhaps being in the sub-micron size. In certain of the fly ashes, many of the particles are of spherical shape.

3. Fly ashes as a group exhibit high puzzolanic activity, as judged

by their ability to combine with lime to form strength-producing compounds. In general, the finer the fly ash and the lower the carbon content, the greater the activity and the greater the contribution to the strength of mortars and concretes.

4. For fly ashes of low carbon content and high fineness, the optimum percentage to be used as a replacement for portland cement under normal conditions of moist curing appears to be about 30.

5. Under conditions of curing such as would normally obtain in mass concrete, for fly ashes of low carbon content and high fineness it appears that there may be definite advantages in using replacements of fly ash as high as 50 percent.

6. In general, it appears that the most favorable results are obtained when the fly ash is blended with portland cements of normal or high fineness and of normal or high-lime composition.

7. On the whole, the results obtained by simply admixing the fly ash with the portland cement are as good as, or better than, those obtained by intergrinding the two materials.

8. Fly-ash cements set more slowly than do the corresponding portland cements, but the times of setting are within the usual specification limits.

The following conclusions refer to fly ashes of moderately low or low carbon content and moderately high or high fineness, blended with portland cements of normal or high fineness and normal or high-lime composition.

9. The water requirement to produce a given consistency of concrete is about the same for fly-ash cements as for corresponding portland cements, and is somewhat less than for most of the natural puzzolans.

10. For percentages of fly-ash replacement up to 30, the compressive strength of concrete cured under standard conditions is somewhat lower for the fly-ash cements than for the corresponding portland cements at the early ages, but is substantially higher at the later ages.

11. When concretes are cured under mass conditions (in sealed containers at 70° F. for one day and then at 100° F.), cements with fly-ash replacements as high as 50 percent exhibit compressive strengths greater than do the corresponding portland cements even at the relatively early age of 28 days. (For 30-percent replacements, more than 1000 p.s.i. higher at the age of 3 months.)

12. By replacing a cement of normal fineness with up to 20 percent of fly ash and by intergrinding this mixture, there may be produced fly-ash cements which at ages as early as 3 days exhibit substantially the

same concrete strength as the corresponding high-early-strength portland cement for which the same energy is consumed in grinding.

13. The contribution (in percent) of fly ash to the compressive strength of concrete is not markedly affected by richness of mix.

14. The modulus of elasticity of concretes containing fly-ash cements is slightly lower at the early ages and somewhat higher at the later ages than that of concretes containing corresponding portland cements, but the difference is not of sufficient magnitude to be a factor in design.

15. For masses of ordinary thickness, such as are normally found in highway slabs and in the walls and frames of buildings, the drying shrinkage at the exposed surfaces of concrete up to the age of one year is for fly-ash cements about the same as, or somewhat less than, that for corresponding portland cements. At a short distance from the exposed surface the drying shrinkage up to the age of one year is substantially less for concretes containing fly-ash cements than for concretes containing corresponding portland cements. For very thin sections and for cements of normal fineness the drying shrinkage of concrete may be expected to be slightly greater for fly-ash cements than for corresponding portland cements. It appears that the drying shrinkage of concretes containing finely ground high-early-strength cements may be somewhat reduced by the use of fly ash.

16. As compared with the corresponding portland cements, the fly-ash cements exhibit about the same or somewhat less resistance to freezing and thawing of concrete.

17. Concretes containing the fly-ash cements are more resistant to the action of sodium sulfate than are concretes containing the corresponding portland cements.

18. The heat of hydration of fly-ash cements is less than that of corresponding portland cements. It appears that the percentage of reduction of the heat of hydration up to the age of 28 days is roughly one half of the percentage of fly-ash replacement.

19. When subjected to sustained load, mortars containing fly-ash cements exhibit greater plastic flow at early ages than do mortars containing the corresponding portland cements.

ACKNOWLEDGMENTS

Acknowledgments for valuable services in connection with the investigations are due to G. E. Troxell, Associate Professor of Civil Engineering; to E. H. Brown, E. J. Garbarini, Alexander Klein, and C. M. Price, assistant research engineers in the Engineering Materials

Laboratory; and to G. E. Dillon and C. W. Thomson, senior students in Civil Engineering, all of the University of California.

Discussion of the foregoing paper will be welcomed if received in triplicate by the Secretary of the Institute by August 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

(Supplement)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . SEPTEMBER-OCTOBER, 1937

Discussion of Paper by Davis, Carlson, Kelly and Davis:

Properties of Cements and Concretes Containing Fly Ash*

C. P. DERLETH†

The paper creates the distinct impression that fly ash is to be regarded strictly as an experimental material. Differing from the puzzolan initially used in the construction of Bonneville Dam, many thousand tons of fly ash have actually been used in a wide variety of concrete structures. Certain of these have been in service and under observation for more than six years.

The paper reporting the results of the investigation made at the University of California is a distinct and valuable contribution to the American literature on puzzolans. A most important result of the author's investigation lies in the development of a specification covering the grade of fly ash satisfactory for commercial use, particularly when relatively large percentages are used as a puzzolanic blend.

These comments are based on extensive field experiences and on results of tests made in many other laboratories of recognized standing. Engineers in charge of laboratories maintained by the Federal Government, various States, Universities, Municipalities, a Sanitary District, Railroads, Commercial Testing Laboratories and by the Portland Cement Association have been interested and have reported the results of tests. The fly ashes employed were from the State Line Station of the Chicago District Electric Generating Corp. and from the Avon, Ohio, Station of the Cleveland Electric Illuminating Co., both of low carbon content and of satisfactory degree of fineness, sold under the trade name Colloy-Pozulan.

---

*JOURNAL, Amer. Concrete Inst., May-June 1937; *Proceedings* Vol. 33, p. 577.

†President, Colloy Products Co., St. Louis, Mo.

The statement in the summary that "concretes containing properly constituted fly ash cements exhibit about the same water requirement to produce a given consistency" is not substantiated. The test evidence indicates that less water is required. Experiences in a wide geographical area with job scale mixes, in which various aggregates were used, some of the mixes well designed others not, some lean others rich, numerous brands of commercial portland cements being employed, invariably showed that the water required for the consistency corresponding to that of unblended portland cement concrete was substantially less when commercially marketed fly ash was used in rational quantities as a partial replacement or blend with the portland cement being used. Many tests reports confirm this important effect of the use of proper grade fly ash.

The statement that "a probable disadvantage is the necessity for adequate moist curing in order to develop and maintain puzzolanic action" requires comment. The union of silica and alumina (also perhaps the iron oxides) of active puzzolans with lime is purely a chemical reaction as differentiated from hydration. The latter is a dissociation of high limed silicates into lower limed silicates, lime being liberated. The puzzolanic action is a chemical combination instead of a dissociation. The reactions occuring during the puzzolanic action therefore follow certain well defined chemical laws. It is true that both puzzolanic action and hydration of cement cease when no moisture is present. When some moisture is present the reaction proceeds. Saturation is unnecessary for either continued puzzolanic reaction or for hydration of cement. If hydration is interrupted by drying, concrete will not thereafter develop its maximum strength. Tests by this commentator indicate that puzzolanic reaction interrupted by drying will, when moisture is thereafter supplied, continue to completion. The Imperial Valley tests by Thomas E. Stanton, Jr. (a discussion, JOURNAL Amer. Concrete Inst., Oct. 1931, *Proceedings* Vol. 28, p. 131) seem to confirm that intermittent moist curing, when desirable moist curing is not continuously possible, will satasfactorily complete pozzuolanic action.

In this and other papers by Davis, Carlson and Associates, there is continued inference that no constituent of a puzzolana other than silica is reactive with lime. Many authorities have conclusively confirmed by tests that alumina in puzzolanas is also reactive with lime in the formation of mono calcium aluminate.

A study of the authors' compressive strength determinations when using various percentages of fly ash brings realization that to secure optimum results under job conditions, the optimum percentage of

blend should be predetermined by the usual trial tests made during the design of the mix. Any portland cement and a known quality of highly active, superfine puzzolan combined during mixing of concrete in predetermined optimum proportion for the particular work, obviously will produce better results than through the use of a premixed cement made for general purposes and having a fixed and conservative ratio of cement and blending material, such premixed or blended cement perhaps containing either a deficiency or an excess of puzzolan for the particular service requirements. No *single* percentage of any blending medium is the optimum for all classes of work involving different conditions of placing, various early service and ultimate strength requirements, and conditions of exposure. Therefore, it seems desirable that separate standards should be promulgated relating to the grade of puzzolans and other blending media, these in preference to a standard fixing the grade of "Blended Cements," the latter produced from such grade blending media with portland cements for which grade standards are now in effect.

In most jobs where fly ash has been used as partial replacement with standard portland cement, the effect of the use of fly ash in reducing the early strength has not been of importance for the reason that the early strengths of most commercial portland cements far exceed those which were yielded by the same brands some few years ago and, for reason that most designs are based on strengths at 28 days. No instance of flash set has occurred and there have been no delays in the removal of forms. It is regretted that concretes leaner than 1:5.6 containing 5.5 sacks of cement per cu. yd. were not included in the series. Experiences have shown equally favorable results in leaner mixes and that workability was particularly increased.

There is but passing mention in the paper concerning the effect of the use of fly ash on workability and the corelation of factors which influence. As compared with unblended portland cement, fly ash has (1) greater absolute volume, and used as a blend to replace percentages by weight, gives the effect of a richer mix; (2) greater degree of fineness, giving the equivalent of a finer cement, and (3) spheroidal shaped particles which effect a reduction in the friction at the aggregate interfaces, all contributing to increased workability. Their combined effect is of such magnitude as to make necessary a reduction in the amount of mixing water, usually within the range of two to five gallons per cubic yard of concrete.

The fly ashes which the authors used in their tests were neither treated nor refined. As a result of extensive research by several

investigators, means have been found for producing improved fly ash blends and/or blended cements. These yield higher early and ultimate strengths, greater resistance to freezing and thawing and reduced volume change. It is suggested that these improved fly ashes be included in any future investigations.

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MAY-JUNE 1937

Experiences of an American Contractor in London, England*

By John G. Ahlers†
MEMBER AMERICAN CONCRETE INSTITUTE

To tell much about construction methods in England is a difficult task as I know very little about how it is done in England. The only thing that can be told is how one building was built in England by an American construction staff. Whether this is similar to the way the British do it or not it is impossible to say; we were entirely too busy trying to do the work in our own way under British conditions and unfamiliar obstacles.

Even before starting for London we were told that it would be most difficult to get work done promptly under British conditions. We were told the same thing more emphatically after arrival in London, and on several subsequent occasions we were ready to agree.

However, inasmuch as two of us started out on four days notice from New York in September of 1935 and arrived in London with a large bundle of plans, a letter of credit and a firm resolve to try to do what others had said could not be done, and returned after the factory was in operation early the following April, it is only fair to say that the British can do as well as we can if given a fair opportunity.

On September 4th we started out with that bundle of plans which by some intuition consisted of four copies of an outline and sketches of a building identical with one that had been built in New York. We returned with another bundle of plans, twice as large, and in no manner resembling the original plans except for dimensions of the outline. The one set of plans that came back were the American drawings corrected and added to by instructions by cable and otherwise, as to how they would be acceptable in England. And another

*Presented at 33rd Annual Convention, American Concrete Institute, New York, Feb. 23-26, 1937.
†Vice-President Fred T. Ley Co., New York; formerly of Barney-Ahlers Construction Corp.

large bundle of plans, detailed drawings required by the London County Council, fully as large and just as bulky as the set of drawings usually needed for the construction of a building in New York. We returned with our new plans and with a profound respect for the British people.

It was perhaps the utter lack of knowledge of how they do things over there that enabled us to get things done, as we did not hesitate to "barge" in and insist on things moving. By the time the job was done we had learned to know the British engineers, the contractors and sub-contractors so well that we would not have the nerve to do again what we did innocently at the beginning of this operation.

An American engineer must have on going to England, a good substantial code book, an infinite amount of patience and an ability to go to bed and to sleep and forget all the worries and difficulties of the day before. I speak again of the code book and the cable because if one is three thousand miles away and has to confer constantly with people at the other end, much time is consumed decoding and composing skeletonized messages. One must be prepared to be called out all hours of the night by telegraph messages, then to pace the floor for an hour or two composing an answer before going to bed again. New York is forever forgetting that when they are sending the last message at night, it is usually after bedtime when these cables are delivered in London.

In many respects England is even more foreign than a country would be where the people speak another language for the very reason that we expect them to be like us. Their language is ours but their ways *seem* the more different because we are less prepared for different ways than in a country where we would expect to be tolerant in the face of a foreign tongue.

Perhaps the greatest factor for success in doing a single construction job in England, is the matter of contact and friends acquired during an undertaking.

Perhaps this is no truer than in an American city; nevertheless the heads of organizations, departments and firms in London, when properly approached and properly introduced, will put themselves out to help a stranger. A British engineer or responsible man in charge is just as amenable to a little flattery and praise as people in the same positions here, and when made to feel that they are expected to do as quick work as is done in New York, they will put their shoulder to the wheel and help the job along. The right friends and the right contacts were invaluable to us, this perhaps especially in securing permits and cooperation from the City officials.

The London County Council is an ancient, autocratic and powerful

body. It has done its work in a certain manner for so many hundreds of years that no foreign arguments or persuasion would ever make it swerve one inch. The head of the Architectural Department, their Bureau of Escapes, their fire brigades, their bureau of engineering are as inflexible as the very peculiar branch of His Majesty's Government, the Home Office, which by ancient authority has jurisdiction over the electric wiring of every building in London.

Arriving in London on Sept. 9, we first conferred with the owners and then straight to the head architect's office. Unfortunately, (or as later proved, fortunately) he was not in and I saw his second in command. My greeting after I had presented my letter of introduction was something like this: "So, you're another American engineer like Mr. ——. Are you, like him, coming over here selling a project, turning it over to the English and walking away with £30,000 in your pocket?" I didn't take up the challenge. Before long we were laughing together and the way was open for a long pleasant contact for the life of the job. A hearing was arranged for Sept. 23; contacts were established giving us 12 days in which to convince the various departments that what we proposed to construct was safe, practical, necessary and not too conflicting with the long, established rules and regulations of the London County Council.

Let it be said too, that at that particular time no information was available as to the character of the soil under the building, except that piles had been used on some buildings on the site and spread footings under others. No survey had been made of the site and no layout as to sanitary drains, electric power installation, etc.

It was proposed to use steel cores in the interior columns as had been done on an identical building in New York, but no building with steel cores had ever been built in London. It seemed for a time that an Act of Parliament might be required.

However, Providence seems to have had a hand in everything that happened and some kind official in the London County Council, who was very sympathetic and understanding, made the excellent suggestion that a permit be asked, not for a building but for a container for a special chemical process. There are ways and ways, even with the London County Council.

I say Providence was on our side as war clouds were gathering all over Europe and the owners had always the interest of the British Empire to bring up, the good of the nation and the helping of Great Britain in general, as a very powerful and effective motive in paving the way for this operation.

Our first contact with British engineers was thus with the pains-

taking, well trained, efficient and thorough-going examining and designing engineers of the London County Council, plus a very understanding and rather practical engineer from Birmington who had been retained by the owner as consultant and designer of the foundations.

When the whole undertaking was made a sort of a contest or a game, it was remarkable how quickly the men fell into the spirit of the play, more so as everyone seemed to be thoroughly infused with the idea that they could do as well or better than the performance an American is likely to boast about.

We had come over to find out what the obstacles were and what information could be secured in order to make the project a success. All of the data had to be cabled back to New York to fit the project into shape here and meanwhile pave the way for the actual construction on the other side. Between September 12 and September 23, information was gathered on steel sash, requirements as to reinforcement and the building code design of foundations and their revision after loads were changed by cable from New York. Borings were made on the site, drainage layout arranged, survey of the property and layout of the proposed building, making computations, fortifying the application to the London County Council, preparing the case for the committee hearing September 23.

September 23 I succeeded in having a personal conference with the Chairman of the Committee, and again here was the challenge and the same jibe about an American coming over selling ideas. This jibe dodged, giving some degree of assurance that we were not there to sell anything, but merely to help a British enterprise, secured the cooperation of this Chairman of the Committee.

There were many walks through many corridors with heavy oaken doors into anti-chambers and conference rooms, ending at the desk of the high power of the department, even the unusual calling in and personal meeting and discussion with the committee as a whole. There was great satisfaction in walking away from this meeting with a verbal assurance and approval of the application.

Now the London County Council does not say: "You must not do so, you cannot proceed with this construction," they merely say that whatever you do is subject to their approval and if you proceed you are taking your own chances. This is perhaps one point where we are more daring than our British cousins and where the owners of the enterprise due to the various conditions and necessities of the moment, caught the spirit and assumed the risk of proceeding without any formal, final approval of any plans or details. The steel cores *were*

accepted as part of "the special container" and these could therefore immediately be designed, purchased and be ready for delivery and erection while the reinforced concrete piles and caps were being put in place.

The foundation contract was let September 28 and on October 3 it was decided by the directors of the owner company to retain us for the period of the job so that we might endeavor to have the project completed in the Spring. This was a big promise inasmuch as the earliest date of completion contemplated by any British organization was nine months later.

From then on a series of conferences—planning, scheduling, selection of subcontractors and the usual routine in an unusual environment, but with the same pleasant extremely civil cooperation that the British have in all their business dealings.

A long telephone conversation gave assurance from London to New York that everything was going fine, and that all that was required was more technical help, more engineers from this side to make the project a success. With all these things arranged on a transatlantic telephone, the job kept going with a minimum amount of deferring to New York for approval, but with a tremendous amount of faith and confidence and trust on all sides, as long as there was a reasonable assurance that the task could be accomplished.

This necessity for redrawing all of the design to meet the exacting requirements of the London County Council became one of the great urgencies of the operation. And in order to do this successfully it was found necessary to get the cooperation of British engineers to draw these plans to the exact requirements as quickly as possible ahead of the construction program. However, showing a mutual faith and confidence it may be said that construction was always several floors ahead of the final approval of the detailed drawings by the London County Council. And only in one instance (the matter of framing around a three foot diameter hole in the floor) was there any serious difference of opinion or doubt on the part of the official as to the work being done to meet their requirements.

Perhaps many years of experience in construction trains one to the necessity for taking chances, but taking these chances with a reasonable assurance of success; if so, this training came in very handy on this operation and the result justified whatever was done. Another difficulty to be overcome was a complete change in loading of some of the columns after the footings were designed, New York having decided to add to the load of one floor, add a tank on the roof, and change the story height.

However, the London County Council would let you come in and make corrections in red ink on the drawings provided they had not been finally approved. So we moved along. In spite of everything the foundations were finished as promised, November 15, making possible the erection of the steel cores shortly thereafter.

We were preparing a bill of quantities and this is no small task under the British method of estimating and contract. Every floor, every item, every piece of work going into the building is itemized and the quantity listed against which the competing contractors enter their unit prices and the contract is based on this tender.

At about this time we ran into one of the ancient, underground passages of London and one of the footings was found to be located in a bottomless pit. In spanning this pit we encountered a tunnel 30 feet below the ground and under the bed of the river, and a special cruciform shaped footing had to be designed.

London blue clay is peculiar material, when wet it is slippery and soft, when dry is gets very hard. It was necessary to find some records of the resistance of this clay to pressure and flowing under load, and excellent cooperation was secured from a young geologist who rooted out the data.

Much time might be given to discussing types of piles. The workmanship of the contractors on this work is excellent. The men are skilled, the equipment on hand no different from ours. It is adequate, and the men are willing to work long shifts, to make a little extra money. It became my duty and responsibility to make a written guarantee against subsidence of the building and about this phase of the work much could be said too. The guarantees were given and on completion it was practically impossible to find any indication of settlement.

Much might be said of design. The steel cores were heavier and stiffer than would be used under our standards.

The final structure in no manner resembled the original plans. I have reference especially to windows, provisions for escape, doors, arrangement of the stairs, the ventilation of the elevator shaft, etc.

Everything was accomplished, however, thanks largely to the splendid cooperation of the owners and the men with whom contacts were made.

The owners' consulting engineers on the mechanical layout of the equipment of the building arrived about the time of the pouring of the floors and their early and late work and their fitting of the process equipment into the structure insured success of the enterprise.

In selecting a contractor for the erection of the concrete framing,

we were exceedingly fortunate in not choosing a "big" contractor—again, in finding an organization anxious to learn as much about American methods as possible and not at all antagonistic.

The low bidder was much below the next one in line and had no experience in reinforced concrete and it was therefore with a great deal of trepidation that I permitted the owners to accept them as the contractor for this part of the work, on the basis, however, that I be permitted to go over with him his entire estimate and break down the items on which I felt he was too low. After doing this, I found many discrepancies and omissions in his tender and after these were adjusted he proceeded with the work. It therefore became a considerable part of my duties to help him in selecting his method of handling the work and the plant to be used.

Having had much to do with plant installation, it was interesting to visit other buildings in London and, as a coincidence, there was at that time an exposition similar to our annual road show, an exposition of construction machinery. Much to my amazement on visiting this show in the company of the contractor, I found that the plant exhibited was very inadequate to our conception of the requirements of a seven story reinforced concrete building. There was no mixer larger than thirteen cubic feet, and there was no hoist, either steam or electrical capable of lifting any greater load than a half a cubic yard. The contractor had contemplated placing about four or five small 7S mixers around the building and hoisting the concrete on the so-called "monkey on the stick" and enclosing the entire building with pipe scaffold, wheeling the concrete around the outside as mortar is handled for brickwork.

The one piece of equipment at the show, however, which everyone was admiring consisted of a unit of a 10S mixer, a chain belt for loading a bucket conveyor which loaded into overhead bins. On these bins were weighing devices for measuring the sand and the gravel and there was also an accurate water measuring tank. The whole unit was dismountable and intended probably more for road work then building work.

Fortunately the contractor took quite a fancy to this piece of equipment and as nothing better was available, I permitted him to use this for the building, having his promise, however, to supplement it with additional 10S mixer on the opposite side of the building for other work. In addition to these two pieces of plant, I had him install one of our standard platform hoists, though no one believed it would ever be of any use for the building. This, however, was one of the saving points toward the end of the job and this hoist, when subcontractors

were working, was in such constant demand that we found it necessary to work it two shifts a day in order to bring materials up through the building.

An interesting comment on cost of materials in London, is that sand is more expensive than gravel and therefore the desirability of oversanding a mix becomes an expensive operation for the contractor, quite contrary to the fact that here, when we oversand the mix, it becomes cheaper.

The cement was tested much as we test it here. but the concrete test is still being made on cubes and I found it impossible to persuade City Authorities, testing laboratory, contractor and others, to use cylinders instead of cubes.

The proportioning of aggregate was well controlled by the weighing device on the charging plant. On this part of it there was no trouble once having arrived at the proportion which was acceptable to myself and still reasonable enough so as to make it acceptable to the contractor.

There are no deformed reinforcing bars in the London or English markets and we had to use plain bars, an Englishman considering a deformed bar as a dangerous innovation in reinforced concrete construction.

A word on winter weather protection. I had insisted in taking tenders to provide for full winter weather protection, though everyone assured me that in the London climate it would be unnecessary. Now that the job is finished, I concede that it was an unnecessary precaution. Nevertheless I had the satisfaction of seeing the building completely closed on two days and salamanders and coke used on one section of the building. During the rest of the winter weather construction (November 15 to February 22) thanks to blanketing of heavy fog, we only had two days on which the temperature dropped below freezing. These fogs are a great inconvenience for construction in the winter time. There were many days on which the sky became so overcast with a combination of smoke and fog that flares and lights had to be used throughout the day. In fact, one day in pouring the second floor it became so dark and so foggy that the men pushing the buggies on the runs on the floor could not see each other and had to be very careful so as not to run off the run panels.

An additional comment on the winter weather protection is interesting in that the British foreman in charge of the work had never heard of heating the concrete by introducing steam in the mixing water. I had the owners supply a steam line and arranged for the contractor to put in a storage tank of water to which the steam line was run. This procedure was very amusing to the contractor. When he saw the concrete arriving at the floor so warm that condensing

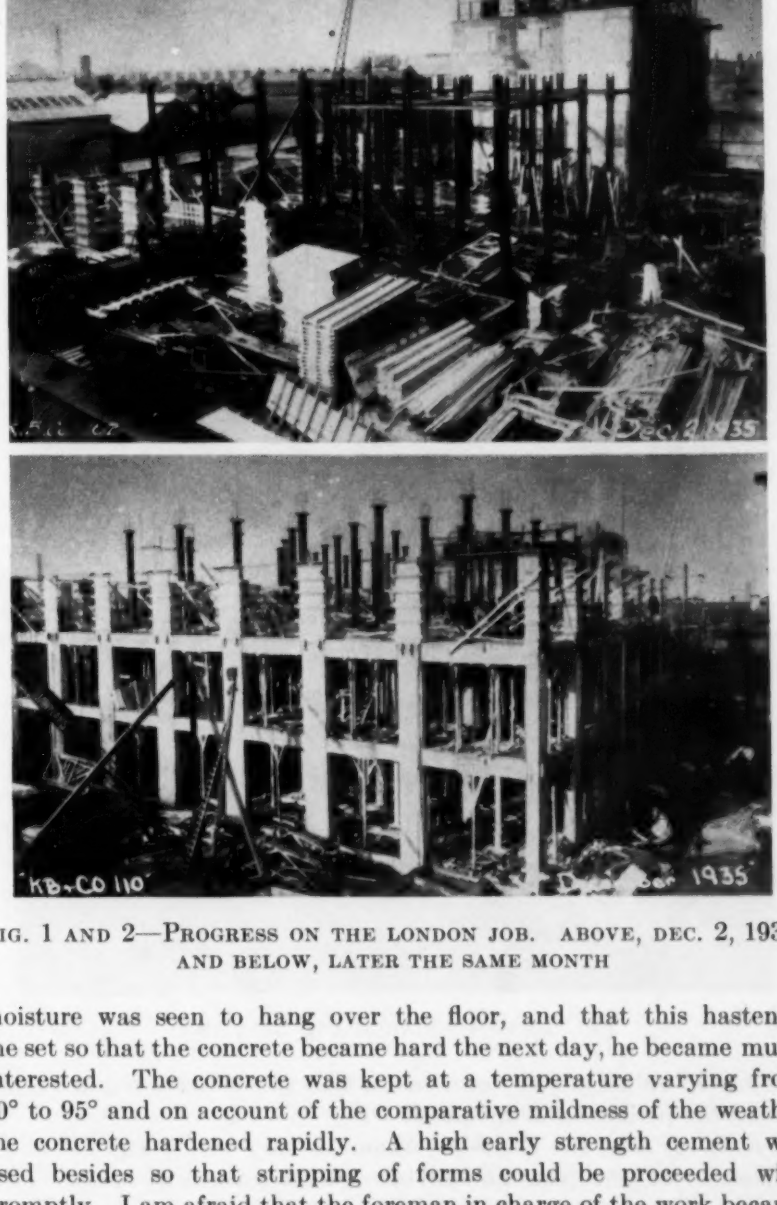


FIG. 1 AND 2—PROGRESS ON THE LONDON JOB. ABOVE, DEC. 2, 1935, AND BELOW, LATER THE SAME MONTH

moisture was seen to hang over the floor, and that this hastened the set so that the concrete became hard the next day, he became much interested. The concrete was kept at a temperature varying from 60° to 95° and on account of the comparative mildness of the weather the concrete hardened rapidly. A high early strength cement was used besides so that stripping of forms could be proceeded with promptly. I am afraid that the foreman in charge of the work became so enthusiastic over this heating of the concrete that he overdid it a little, particularly on the start when the water was very hot and he also

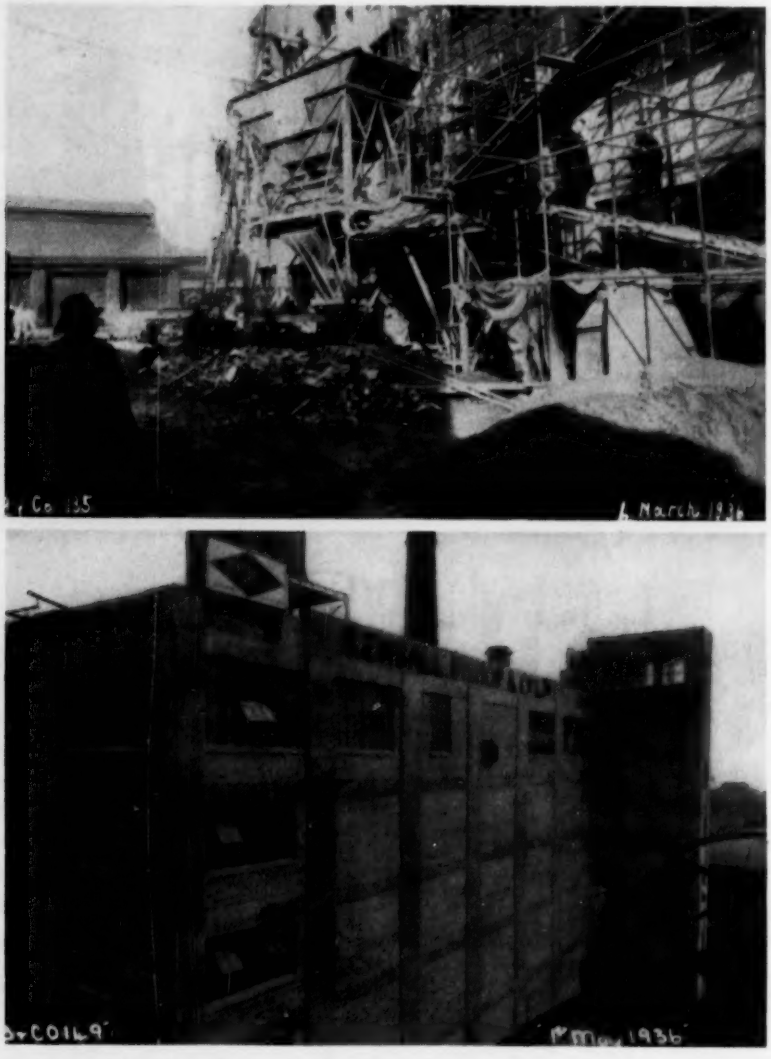


FIG. 3 AND 4—MARCH AND MAY 1936

had heated up his gravel—so much so that the concrete began setting around the perimeter of the mixer.

I must say that the English workman is very untidy and not at all as likely to keep his plant and job in as good condition as we are. This cleaning of the mixer was therefore neglected and the contractor paid a heavy penalty in the smaller output of the mixer and slopping and wasting material until the matter was pointed out to him and he had

FIG. 5—THE GENERAL SCENE OF THE JOB

to make a shutdown over Christmas to clean the mixer out thoroughly. After that it was almost a two-man job to get him to heat his concrete again as he was constantly afraid of a repetition of the concrete setting in his mixer and thereby spoiling his plant output. We used thermometers to check the temperature of the concrete both in the concrete buggies and also after it was placed on the floor.

The finest piece of material and workmanship on the job was the installation of a great quantity of glazed white tile. This white tile was of an exceptionally good quality being made out of the same material from which china and plumbing fixtures are prepared. It was burned at a very high temperature and had a beautiful finish. The men who erected the tile partitions were skilled mechanics who had followed this particular line of work all their lives and the contract was let for both furnishing of material and erection. This was done by the well-known Doulton & Company, manufacturers of the Royal Doulton China, of which a large part is the industrial china for chemical plants, insulators, etc. They set aside a special part of the plant for the manufacture of these tile and did a beautiful job, though I doubt if they made any money on the contract.

The brick work was of a very good quality. "Flettons" are a very tough, hard-burned brick used in London for inside work and so uniform that we selected them for the outside brickwork and on completion they made a very neat appearance.

Another outstanding subcontractor was the man furnishing the ironmongery, the British name for miscellaneous iron. They are very skilled in this trade, capable of doing fine shop work, even though slow in their field work, using somewhat antiquated methods in the nature of hoisting, hand winches, tripods, etc.

The acidproof floors were laid with material imported from the United States and the foreman in charge of this was a mechanic sent over from the United States.

I arranged for the contractor on the reinforced concrete work to use one of our American superintendents and one of our American carpenter foremen, and these two men fitted into the organization and provided the necessary stimulus to make an expeditious job.

In the beginning of the work the contractor has the opportunity to state how much bonus or penalty he would accept and decided 25 pounds sterling a day. I had told him earlier that he should make it higher as the owners were anxious to get the building but he was afraid of over-running his time. When he finished thirty days ahead of time, he told repeatedly he wished he had followed my advice and doubled or trebled the amount of bonus and penalty.

A word about the City Department; the official control of building work in London falls into two divisions—that of the London County Council and the so-called District Surveyor. The London County Council, similar to our Building Department, has jurisdiction over the plans and permits and the keeping of the permanent records. The District Surveyor is an elected official and has charge of the work in the district in which it is located. His function is to carry out the instructions by the London County Council and is responsible for fulfilling the requirements of the filed plans. The man on this job had some knowledge of construction, and we spent many interesting hours discussing the design of the girders, the checking of cubes, etc., and in the end he was quite willing to accept our statements.

The contractor earned a bonus on thirty days; succeeded in having the concrete placed on the roof by February 22nd and the building substantially completed by March 5. The installation of plant and equipment for manufacturing purposes followed so closely behind that the plant was put in actual operation on April 4, less than seven months after we landed on the other side.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by August 1, 1937. For such discussion as may develop readers are referred to a Supplement to be issued with the JOURNAL *for Sept.-Oct., 1937.*

Vol. 33 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . MAY-JUNE 1937

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers.

New developments in the concrete field

VIERI SEVIERI, *Tonindustrie Zeitung*, Vol.61,No. 27, p. 300-1, Apr. 5, 1937. Reviewed by A. E. BEITLICH

Recent investigations on concrete carried out by physicists and engineers have shown that greater progress can be made in the manufacture and preparation of concrete by making use of certain processes and developments than can be expected from pure chemical cement research.

New developments in the cement field

VIERI SEVIERI, *Tonindustrie Zeitung*, Vol. 61, No. 17, p. 197-8, Mar. 1, 1937.
Reviewed by A. E. BEITLICH

The author gives a brief illustration of the studies of several European investigators in connection with the development of new special cements designed to meet certain specific requirements. The relation between fineness, heat of hydration, and cement constitution and their effect on strength and shrinkage are discussed.

The influence of heat of hydration of the cement on the quality of cement

A. BERECZKY and A. KOCH, *Zement*, V. 26, No. 6, Feb. 11, 1937, p. 87-90.
Reviewed by L. T. BROWNMILLER

The time-temperature relations were measured during the hardening of concretes of two portland cements, one super portland cement, one slag cement and one alumina cement. The results only show the course of the development of the temperatures. No sufficient data are reported to draw any definite conclusions regarding quality of concrete.

Properties of the mix and cement content of mortar and concrete

A. STEOPOE, *Tonindustrie Zeitung*, Vol. 61, No. 11, p. 138-9, Feb. 8, 1937.
Reviewed by A. E. BEITLICH

The author comments on the proposed German specification for the determination of the proportions in hardened mortar and concrete. He points out the necessity for taking into consideration the size of the aggregates when selecting a representative sample for analysis. He also cites results of tests in which the proportions were found by determining both the soluble silica and the soluble calcium oxide. Good agreements were obtained under certain local conditions.

Severe damage to outfall sewer caused by action of acids

Engineering News Record, Vol. 118, No. 14, Apr. 8, 1937, p. 519-520.
Reviewed by S. J. CHAMBERLIN

An inspection trip through the north outfall sewer in Los Angeles showed that much of the cement mortar was destroyed, and that there was cracking and spalling of the tile liners. The outfall is an unreinforced, semi-elliptical concrete sewer, lined with clay blocks. The city engineer's staff reported that the disintegration is the result of sulphuric acid attacks on the portland cement mortar joints between the tiles. The sulphuric acid was formed by the oxidation of hydrogen sulphide gas in the presence of water. The acid softens the cement and reduces it to gypsum, which has a much larger volume than the cement.

Water and crude oil-resisting concrete coatings

P. N. GRIGORJEFF and S. BARGTELL, *Tonindustrie Zeitung*, Vol. 61, No. 10, p. 127-8, Feb. 4, 1937.
Reviewed by A. E. BEITLICH

Effective coatings for concrete tanks which are not attacked by water, fuel oil, or gasoline are recommended. These have been tried in laboratory tests and in practice. A paste prepared from 24.2 parts of artificial varnish, 33.3 parts of hydrated lime, and 42.4 parts of clay or other inert filler hardens in 7 to 8 days when spread in a thin layer. The second formula suggested for crude oil consists of 7.8 parts of portland cement, 59.4 parts of finely ground clay, 2.0 parts of sodium fluosilicate, and 37.4 parts of water glass (38°). The second mixture hardens very fast within several minutes and is applied in 1 to 2-mm. thick coatings. Most of the reaction products are relatively insoluble.

The microscopic study of cement in reflected light

ERICH STACH, *Tonindustrie Zeitung*, Vol. 61, No. 28, p. 318-19, Apr. 8, 1937.
Reviewed by A. E. BEITLICH

A new method, developed by the author, consists of a microscopic study of powdered cement embedded in a black rosin which is ground and polished. The specimen is observed under the microscope with the oil immersion objective. The method has several advantages. It allows a study of very fine grains and the distribution of compounds in various grain fractions. It is possible to study cements from which clinker samples are not available. It can be used for examination of cement raw mixture at various stages of heating. Mixed cements can also be examined, and finally it is hoped to study various stages of hydration of cement if it is possible to find a medium in which the powders can be embedded without the application of heat.

Rapid method for the determination of calcium oxide and magnesia

HORNKE, *Tonindustrie Zeitung*, Vol. 61, No. 28, p. 311-12, Apr. 8, 1937. Reviewed by A. E. BEITLICH

Experiments are described which deal with methods for rapid determination of CaO and MgO. The determination of CaO by titration of calcium oxalate in sulfuric acid with potassium permanganate gives accurate results if the solution is heated to 70° C The presence of filter paper does not interfere. It is shown that the precipitation of MgO as magnesium ammonium phosphate is complete in 30 minutes and qualitative in boiling solution followed by agitation for 30 minutes. The presence of oxalate ions causes some retardation in the precipitation. The titration of phosphoric acid gives accurate results if it is over-titrated with hydrochloric acid and the excess acid titrated back with sodium hydroxide.

The present status, purposes and usefulness of applied soil investigations

N. Loos, *Beton u. Eisen*, Vol. 36, No. 5, p. 77, Mar. 5, 1937 Reviewed by A. U. Theuer

This is a descriptive rather than an analytical treatment of the problems involved in soil mechanics and foundation design. A historical sketch mentions the sources of much of the early work and enumerates the many limitations that are still encountered in all attempts at practical application. The results obtained to date are stated to be largely qualitative, but of considerable practical value in spite of this serious limitation. Factors to be observed and proper procedures to be followed in soil-foundation investigations are described in detail together with the special demands made upon an engineer in charge of foundation studies. The article is copiously illustrated.

Hot weather concreting

Engineering News Record, Vol. 118, No. 11, Mar. 18, 1937, p. 414. Reviewed by S. J. Chamberlin

At the Imperial Dam on the Colorado River, temperatures during the summer are so hot that normal operation of concreting was not thought advisable. Temperatures occasionally reach 118° F., but the maximum permissible temperature of the concrete at the point of placement has been set at 85° F. A well supplies water at a temperature of 70° F. This water is used for mixing, and is run through the mixer to cool it off before operations begin. Freshly washed gravel is used if possible or if taken from the bins the bins are cooled by wrapping them in burlap, continuously sprinkled with water. The forms and reinforcement are sprinkled before the concrete is placed. The 8-in. pipeline used in delivering the concrete is covered with burlap and sprinkled its entire length.

The danger of rusting of reinforced concrete structures following the formation of cracks

Paul Abeles, *Zement*, V. 26, No. 7, 8, 9, Feb. 18, 25 and Mar. 4, 1937.
Reviewed by L. T. Brownmiller

The extent of rusting of the reinforcement of concrete was studied in some actual structures and also in a series of experimental structures. The quality of the concrete is of greatest importance in preventing rusting. The cracks formed by loading the structures are never as detrimental as shrinkage cracks, etc., resulting from poor concrete. Rusting may occur in porous concrete even if no cracks are present. Accordingly by suitable design of the concrete and of the proper reinforcing steel for load distribution, the static cracks which form are not wide enough to be dangerous as long as the structure is not exposed to undue chemical agents such as fumes, etc.

New method for measuring the workability of concrete

J. Faury and A. Lamare, *Annales de L'Institut Technique du Batiment et des Travaux Publics*, Vol. 2, Jan.-Feb. 1937, p. 48. Reviewed by P. H. Bates

The test, for concretes having a maximum size of aggregate of 25 mm., consists of tamping the concrete in two cubical molds, 20 cm. on an edge, one containing reinforcement and the other without. The tamper consists of 9 vertical rods, 14 mm. in diameter, loaded to weigh 3.5 kg., and oriented so that it falls freely into the concrete. Two layers of concrete are each tamped 4 times, the fall being 25 cm. for the first and 35 cm. for the 2nd layer. The ratio d/d of densities obtained in the mold with, and in that without reinforcement, is taken as a measure of workability. The sand-gravel ratio, s/g, was varied from 0.35 to 0.70. With an increase

in w/c from 0.60 to 0.85, d/d increased from 0.933 to 0.997, while the optimum s/g increased from 0.47 to 0.52. For values of s/g above or below the optimum, the rate of change in d/d with change in s/g increases as w/c decreases.

Hollow concrete rigid frames support auditorium roof

WILLIAM T. WRIGHT, *Engineering News Record*, Vol. 118, No. 15, Apr. 15, 1937, p. 560-562.
Reviewed by S. J. CHAMBERLIN

A hollow-girder roof has been built for an auditorium at Placentia, Calif. The hollow girders and hollow columns form a series of five rigid bents, having a span of 77 ft. The girders have thin webs of varying thickness and 6-in. top and bottom slabs. The top slabs are cast integral with the intervening roof slabs to form a horizontal diaphragm to transmit seismic forces through the walls to the foundation. The girders vary in depth from 5 ft. at midspan to 4 ft. at the ends, and have an over-all width of 4 ft. 7 in. The frames are carried on continuous footings running the full length of the side walls. The top of the girders are flush with the top of the roof slab, and the roof between girders is carried by concrete joists. The fundamental aim of the design was to unite the top and bottom webs of the girders by means of numerous transverse diaphragms, four interior fillets, and continuity of reinforcement. The unusual framing promises economy, earthquake resistance, and effective architectural appearance.

Restraining moments in the transverse floor beams of two-walled truss type bridges

R. BORTSCH, *Beton u. Eisen*, Vol. 36 No. 5 p. 89, Mar 5, 1937; Vol. 36 No. 6 p. 107, Mar 20, 1937.
Reviewed by A. U. THEUER

A complete mathematical treatment of the problem of restraining moments in the transverse members of the floor systems of girder or beam bridges (two-walled, trough-type) is contained in this paper. The members being statically indeterminate, the problem becomes involved due to many equations. Unitary expressions to describe the influence lines of the bending moments are also derived. Several desirable features of design in this type of bridge are given in conclusion.

Aggressive agents and hydraulic admixtures

A. STEOPOE, *Zement*, V. 26, No. 11, Mar. 18, 1937, p. 169-172. Reviewed by L. T. BROWNMILLER

Trass and other siliceous materials are often added to cements in order to increase their resistance towards aggressive agents. According to the author these materials react with the $Ca(OH)_2$ of cement to form calcium hydrosilicates. The hydrosilicates are not stable and the better durability of trass cements is not due to the presence of the calcium hydrosilicates as such. They are decomposed by aggressive agents, even by water, leaving silica gel which is deposited in the pores of the concrete making it denser and increasing durability thereby. The calcium hydrosilicate is therefore only an intermediate product formed after the silica of the trass has been activated by the calcium hydroxide of the cement.

Author claims that the ordinary methods for determining the amount of free lime in set cement always give values which are too high. Even when hydrated cements are dried at relatively high temperatures, some water remains in them which is dissolved by the lime solvent (alcohol, glycerine, etc.) causing further breakdown of the calcium silicates, etc., with the resulting values for the free lime too high.

Building control during winter concreting

ALWIN GOERTZ, *Beton u. Eisen*, Vol. 24, p. 397, Dec. 20, 1936. Reviewed by A. U. THEUER

According to the author, concrete construction during freezing temperatures has not been generally adopted in Germany. Despite the extensive practice of winter concreting in the more northerly countries, actual experience with these methods is lacking among German engineers who fear not only the technical problem presented but also the increased costs that may be involved. The author describes in detail his own experiences in the construction of a highway overpass during the winter of 1935-36. Overtaken shortly after the commencement of concreting by a period of freezing weather, portable boilers were installed on the job and by a combination of pre-heating of the aggregates and mixing water, insulation of the freshly constructed walls and artificial heating of the structure itself, work was continued.

Throughout the work and up to the time the concrete had attained normal strengths as indicated by test cubes, a close check was kept on the temperature of the walls and slabs by means of thermometers placed in open wells. These data are presented by means of tables and graphs. Stress is laid on the need for special inspection in winter concreting. For this particular job, relatively small, the increase in cost was estimated to be 5 percent of the total cost of the rough work.

Exterior walls of concrete house completed in one day.

Engineering News Record, Vol. 118, No. 11, Mar. 18, 1937, p. 403-404.
Reviewed by S. J. CHAMBERLIN

The use of a vacuum process for pulling out the excess water in freshly poured concrete walls was recently demonstrated in the construction of a bungalow near Washington, D. C. The exterior walls consisted of two 3-in. slabs separated by a 6-in. air space. The exterior forms were built of 4 x 8-ft. plywood panels, lined with a coarse mesh cloth to permit uniform suction over the face from the cups and to provide a passageway for the water. Thin strips of rubber sealed the edges of the panels. Two horizontal strips of rubber divided the panel into three suction areas corresponding to the vertical lifts of the concrete. The center form for the air space consisted of building paper attached to wooden studs. As soon as the vacuum was applied, the paper forms were pushed in against the concrete by the atmospheric pressure. Upon removal of the forms a stucco surface was applied to the exterior before the final set occurred. The 7-in. slump concrete was wheeled to the forms in buggies and dumped into two-spout hoppers that delivered to both wall slabs simultaneously. Electric vibrators were used on the plywood forms. The entire wall was poured, forms stripped, and the exterior wall coating applied in one working day.

Experience with light aggregate in concrete construction.

RALPH S. MCLEAN. *Engineering News Record*, Vol. 118, No. 13, Apr. 1, 1937, p. 484-485.
Reviewed by S. J. CHAMBERLIN

Light-weight aggregate was chosen for the concrete in a building for the Fullerton Junior College in California because of poor foundation conditions. The aggregate was made from low-grade diatomaceous earth, containing a small amount of clay. Oil was sprayed on the crushed raw materials, and the mixture burned in a kiln at a temperature just below that at which clinker would be formed. The loose aggregate weighed about 40 lb. per cu. ft. and rapidly absorbed water up to the 30 per cent point. The aggregate entered the mixer in a saturated surface-dry condition. Considerable amounts of fine material were added to the artificially made ingredients. The resulting volumetric proportions were: cement, 1; silt, 0.39;

natural sand, 0.78; light weight sand, 0.78; light weight pea gravel, 0.56; light weight coarse aggregate, 1.69; water, 6½ gal. per sack of cement. The weight per cu. ft. of the concrete as it left the mixer was specified to be not more than 115 lb. The weight with age is expected to be about 109 lb. Strengths of 2,000 to 3,000 p.s.i. were obtained. Saving in weight justified the increased cost per yd., which was $9.00, compared with $5.89 for natural aggregate concrete on the same job.

The action of alkali-chlorides on cement and concrete

F. HUNDESHAGEN, *Zement*, V. 26, No. 7, 8, Feb. 18 and 25, 1937, p. 103-106 and 117-120.
Reviewed by L. T. BROWNMILLER

Ten grams of cement were treated with 200 cc. distilled water, with 200 cc. 2N and 200 cc. saturated NaCl and KCl solutions for 14 days at 17½° C. The reaction was followed by analyses of the solutions. Eight different cements were used in the studies, viz: three portland cements, two slag cements, one ore cement and two alumina cements.

The reaction was quite characteristic for the different types of cements. In the presence of calcium hydroxide, the chloride is sometimes combined as oxychloride and sometimes as calcium chloraluminate in the residue. Presumably the alkali is present in the form of insoluble zeolites.

The chloride solutions extract large amounts of alumina from the aluminate cements and it is believed that this extraction causes deterioration of the latter cements. The chloride solutions do not seem to dissolve silica.

Concretes behave differently towards the alkali chloride solutions depending on whether they are freshly prepared or whether they have been carbonated. Carbonation of the concrete exerts a protective action against chloride attack which is of great importance in the erection of structures for sea water exposure, silos, etc.

Reinforced concrete tanks for the storage of light mineral oils

HELGE HARBOE, *Beton u. Eisen*, Vol. 36, No. 46, Feb. 20, 1937, p. 66. Reviewed by A. U. THEUER

Reinforced concrete tanks suitable for the storage of light mineral oils such as gasoline, kerosene, benzine, alcohol, etc., are described. Varying in detail but utilizing the same principle for effecting a seal against losses by leakage and evaporation, six different types of structure are described and illustrated. Hollow walls filled with water prevent circumferential losses while a sheet of water over the roof prevents evaporation losses. The stored fluid floats either directly on a sheet of water or is carried by a false bottom over a sheet of water. In their simplest form the structures consist of one tank floating inside of another. The author states that a complete seal was obtained with test containers under 7.5 *m* head of gasoline. In a comparison of the reinforced concrete as against the steel tank it is stated that, in the ordinary type of steel construction, evaporation losses range from 2 to 6 percent per year, amounting to 20 to 60 percent of the first cost of the tanks. Modifications to prevent this loss in steel tanks, such as the Wiggins System, increase their cost by 50 percent. Because of these factors, the cost of the concrete type of tank appears favorable. Another advantage claimed for concrete tanks is the greater likelihood of bridging of their bottoms when built on soft foundations. A four-tank installation used for the storage of benzine, benzol, petroleum and varnish-benzine (lackbenzine), and now in service for over three years, is also described. The invention of the type-construction is claimed for the author and F. Zeemann and stated to be protected by Brazillian patent.

Clinker compounds and their investigation

H. E. Schwiete, *Tonindustrie Zeitung*, Vol. 61, No. 28-31, p. 309-11; 328-9; 341-4; 353-5; Apr. 8, 12 15, 19, 1937. Reviewed by A. E. Beitlich

In the first section of this paper Schwiete describes the work of various investigators on the study of the systems CaO-Al_2O_3-SiO_3, CaO-Al_2O_3-Fe_2O_3-SiO_2, etc. Slow cooling of a mixture corresponding to portland cement composition in the field CaO-Al_2O_3-SiO_2 produces at equilibrium the minerals $3CaO.SiO_2$, $2CaO.SiO_2$, and $3CaO.Al_2O_3$. Rapid cooling leads to $3CaO.SiO_2$, $2CaO.SiO_2$, and glass, with solidification at 1458° C. In the quaternary system CaO-Al_2O_3-Fe_2O_3-SiO_2, the melting point of the glass is lowered by approximately 120° to 1338° C. by the addition of Fe_2O_3, and on slow cooling the $3CaO.SiO_2$, $2CaO.SiO_2$, $4CaO.Al_2O_3.Fe_2O_3$, and $3CaO.Al_2O_3$ are formed, while rapid cooling yields $3CaO.SiO_2$, $2CaO.SiO_2$, and glass.

Part II of the article describes briefly the various compositions of clinker minerals and their terminology. The third section deals with the opt cal properties and their identification in thin sections and powders. Part IV contains a large table listing the fundamental data for the various clinker minerals and shows photo-micrographs. The latest developments in the methods of studying etched polished clinker sections in reflected light are cited.

Section V reports on the application of X-ray technique for clinker studies. The sixth and final section is devoted to chemical analysis. It is shown how the calculated compound composition can vary with the fluctuations in the chemical analysis due to discrepancies of the results obtained by various analysts. The method of calculation by Bogue and the latest one developed by Lea and Parker are described and discussed.

The granulating of cement raw feed without application of pressure

Erich Schirm, *Tonindustrie Zeitung*, Vol. 60, No. 98-102, p. 1207-10; 1224-6; 1237-8; 1247-9; 1260-2; Dec. 7, 10, 14, 17, 21, 1936. Reviewed by A. E. Beitlich

Contrary to processes where the raw feed is formed into briquets of certain shape and dimensions, this paper deals with methods in which the raw feed is packed loosely together in small ball-shaped particles. The shape of the mass depends on the character of the material, water content, admixtures, and the mechanical process used. The method is used mostly in rotary kiln installations. The small balls do not lose their shape even on drying in the kiln due to the presence of clay material in the raw mixture. Usually a simple rotating drum serves the purpose for forming the kiln feed. Dimensions, speed of rotation, pitch, etc. must be designed to fit individual requirements The addition of water may be done either in the drum itself, or in the screw feeding the drum. A number of designs are illustrated with detailed description of their special features One of the chief difficulties is the formation of crusts at the wall of the drum. This can be eliminated by installing scrapers. If the fuel for burning is to be mixed in with the kiln feed it is advisable to add the fuel after the first stages of ball formation. Balls of uniform size can be made by screening out the fines and returning them to the drum. A number of designs are described. Sometimes it is desired to produce small balls of considerable hardness. This may be accomplished by adding certain admixtures such as hydrated lime, which hardens under the influence of CO_2, of kiln gases, or steam which is passed through the drum. This process is recommended in the case of raw feed consisting of limestone and blast furnace slag. Finally, suggestions are made to apply the method in wet process plants.

Large bridges of reinforced concrete—(the Roche-Gyon bridge)

VALLETTE and ESQUILLAN, *Annales de L'Institut Technique du Batiment et des Travaux Publics*, Vol. 1, No. 6, p. 36-50, 1936 Reviewed by D. E. PARSONS

After giving a brief history of developments in the techniques of designing and constructing large bridges, M. Vallette outlines a systematic method for selecting the most economical design for a reinforced concrete arch of long span. In particular, he considers the effect of the shape of the arch axis, the dimensions and shape of the arch ribs and the working stresses. Three graphs are shown to illustrate the effects of changes in these variables on the cubical contents of the arch ribs. The first shows the quantities required for arches having a span-rise ratio of 5 with spans ranging up to 3000 ft. and for several working stresses ranging from 700 to 3500 p.s.i. The second gives the relation between volume and span-rise ratio for two different combinations of stress and span, and the third between volume and stress for several spans ranging between 75 and 2500 ft.

The article by M. Esquillan describes briefly the design studies for selecting the most favorable type of arch for the Roche-Gyon Bridge and the construction of the bridge. The arch has a span of 529 ft. with a span-rise ratio of 7. The deck is suspended from the arch ribs on vertical hangers of reinforced concrete within the central ¾ of the span. The arch ribs were of unique design. In order to vary the moments of inertia of the ribs in accordance with the optimum law, both the shapes of the sections and the disposition of the reinforcement were varied with distance from crown. Both the width and the depth of the ribs were varied; portions were solid, the rest was hollow. The highly stressed part of the rib contains hook reinforcement as well as longitudinal bars. The article describes also the methods of erecting the arch centers, the placing of the reinforcement and concrete and the details of pre-stressing the ribs by the Freyssinet method.

Reinforced asphalt concrete

OTTO COLBERG, *Beton u. Eisen*, Vol. 36, No. 5, p. 86, Mar. 5, 1937. Reviewed by A. U. THEUER

An experimental study of the physical properties of a new kind of cement, the discovery of which is credited to Benedikt Carpzow, with the object of determining its suitability for use in reinforced concrete sections, is described. The cement consists of mixtures of Plankton-slime, a material forming a nuisance in many German harbors and requiring periodic removal, and an asphalt or bitumen. The two materials are mixed at slightly raised temperatures and then permitted to cool. Hardening or "set" is said to take place almost immediately, and considerable strength is attained after a few hours. The chemical analysis of a typical slime found suitable for use in such a cement is given in the form of a table. The maximum and mimimum strengths of prisms made up with the standard sand and cements having different proportions of slime and tar were found to be as follows: at the age of three days, compression 182 to 234 kgs/cm², tension 15 to 39 kgs/cm²; at the age of 14 days, compression 249 to 322 kgs/cm², tension 27 to 43 kgs/cm². The actual proportions of slime and tar in the different cements are not given. Strengths of concretes with their aggregates proportioned for maximum density and with a maximum aggregate size of 15 mm. fell within the following extremes: Series A, age 24 hours, compression 368 to 445 kgs/cm²; Series B, age 5 hours, compression 463 to 613 kgs/cm², tension 58 to 68 kgs/cm². Beams 10 by 10 by 68 cm. made up of this same concrete gave moduli of rupture ranging from 52 to 106 kgs/cm². Very serious plastic flow was noted in the case of these plain concrete beams. Supported at their

ends for a period of six months the beams slumped visibly at the center under their own weight. A conjecture arose in the mind of this investigator regarding the use of this type of concrete in conjunction with reinforcing steel. Bond tests, which are sketchily described in the paper, were sufficiently encouraging to attempt additional work on actual beams. Four beams tested to failure under a single concentrated load are described and illustrated. Using 3.1 percent of reinforcing steel, moduli of rupture of 44 to 345 kgs/cm² were obtained. Both the kind of tar and type of slime in the cement were varied. A beam reinforced for both shear and tension was also tested yielding a modulus of rupture of 440 kgs/cm². Further tests are to be reported at a future date.

Large concrete shell roof covers ice arena

ANTON TEDESKO. *Engineering News Record*, Vol. 118, No. 14, Apr. 6, 1937, p. 505-510.
Reviewed by S. J. CHAMBERLIN

An impressive example of a thin concrete shell roof has recently been completed in Hershey, Pennsylvania. The barrel type roof is of the Zeiss-Dywidag design, only 3½ in. thick (except at the edges) and is supported by two hinged concrete arches. The roof, covering an area 232 ft. wide by 340 ft. long, is built in five units separated by four expansion joints. In the two end units, 52½ ft. long, the roof barrel spans 32 ft. from the end wall to the first arch and then cantilevers 19 ft. 7 in. In the three intermediate units, each 78 ft. 4 in. long, the roof barrel is supported by two arches, 39 ft. 2 in. apart, and cantilevers 19 ft. 7 in. beyond each arch. The arches are 22 in. wide by 60 in. deep at the crown, increasing in depth toward the springing line. The theoretical span is 222 ft., and the rise is 81 ft. Plain 1½-in. diameter bars reinforce the arch near the crown and near the point of maximum negative moment. Special hinges are used, consisting of soft lead plates in combination with Mesnager hinges. Bars 1 in. in diameter, small enough to take the bending stresses due to the rotations under extreme loading conditions of 24 minutes, were chosen for the hinges. The reinforced concrete shell was poured monolithically with the arches and thus is restrained at the arches as well as by the longitudinal edge members. The shape closely approximates an ellipse. The curved shell is structurally similar to a girder of 39-ft. span, and having a high sectional modulus serves as lateral bracing for the arches. In addition, it acts as the compression flanges of a T-beam. Where the shell is under membrane stress condition only the reinforcing is placed in the direction of the stress trajectories to take up the principal tensile stress. The principal members and the shell were designed for a 3,000 lb. concrete. The actual strength of the 1:2.75:2.25 concrete was 4,000 p.s.i. The water cement ratio varied between 0.65 and 0.80. The concrete was cured for one week and the centering remained in place from 10 to 17 days. Deflection tests were made on beam specimens of the shell concrete to check its elastic properties, a minimum modulus of elasticity of 3,000,000 p.s.i. at 28 days being specified. The actual deflection of the arches when decentered was within 5 per cent of the calculated 1¼ in., which is 1/2,000 of the span.

Future problems

F. V. EMPERGER, *Beton u. Eisen*, Vol. 36, No. 6, p. 102, Mar. 20, 1937 Reviewed by A. U. THEUER

F. V. Emperger in a contribution to the current discussions in "Future Problems," criticises concrete specifications in their failure to make available to practice with reasonable promptness the disclosures of investigations in the form of practical recommendations. Conclusions drawn from investigations, new developments, and

improved methods of analysis appearing in current literature rest in print awaiting the resurrection of the next general code revision committee. This customarily takes place but once every five or ten years in most countries. And then, according to the author, the cumbersome time-consuming procedure of a general revision frequently passes over important developments in the maze of debate centering on details of a formal nature. The cases of Austria and Switzerland are cited where three and five years respectively were required to effect their last general revisions. The delegation of authority by the general committee to a subcommittee to study and recommend specific changes, as was done recently in Germany, is considered as a step in the right direction.

The second part of this contribution advances 13 specific proposed revisions of the German specification for reinforced concrete. The first of these would require a clearer recognition of the field for compliance. Many of the elaborations of the present specifications are applicable only to large engineering structures, and unless a clear-cut distinction is made the legitimate use of concrete in small structures will be unnecessarily limited. Nine of the other proposed revisions deal with the design of beams. The first of these, which is discussed at length, concerns the use of higher working stresses in members wherever the reinforcement is such that failure will depend only on the strength of the steel. An interesting method of design is advanced wherein an allowable concrete working stress does not enter into the computations and it only remains to determine the maximum amount of reinforcement and the working stress for a given minimum concrete strength.

The remaining proposals are not discussed. In closing, the author cautions against the inclusion of the viewpoint in specification work that concrete design and construction is a completely finished science, and thus limit all creative work to minor details.

Centrifugally spun concrete beams

FRITZ V. EMPERGER, *Beton u. Eisen*, Vol. 36, No. 2, Jan. 20, 1937, p. 25. Reviewed by A. U. THEUER

The author presents a general discussion of centrifugally-spun hollow concrete beams used as elements in building construction. A brief description and an enumeration of the advantages offered by this type of structural element is followed by a detailed criticism of the requirements of the Austrian standard specifications governing design and permissible loading. The basis of design, i.e., that the concrete above the neutral axis only be considered as effective and the added tensile steel carry the entire tension, is considered to be unjustified in the design of this type of beam. A network of steel consisting of radial loops, and longitudinal bars, in addition to the added tension steel is here used, and moreover, the piping results in a three dimensional stress problem quite different from the conventional rectangular beam. For the interpretation of test data and a better theoretical treatment of the problem, the author considers the breaking moment to consist of two separable parts. One part would be the moment as derived by the specification (conventional) method. To this an auxiliary moment $Mo = \frac{bd^2}{6}\sigma_{ct}$ (where σ_{ct} = allowable tensile stress in concrete) dependent principally on the concrete would be added. The total moment as expressed for purposes of interpretation of the results derived from tests made on beams both with and without additive tensile reinforcement would follow from the expression

$$M_{bm} = A^s\left[d - \frac{kd}{2}\right]\left[\sigma_s + \Delta\sigma_s\right]$$

whereby the value of $\Delta\sigma_s$ or the additive moment would be determined. On the basis of this formula a series of tests (made by Dr. Kral in Larbach, Austria) of hollow beams is presented by means of a table. Increases in the breaking moment ranging from 56 to 123 percent were found. In these tests, which are described briefly, the variables consisted of different types of steel, different aggregates and varying cement-aggregate proportions. The auxiliary moment was found to be especially large in the case of beams with very light reinforcement. It was further found that with a reinforcement of 3.06 percent and a computed concrete stress of 400 to 500 kg/cm², the steel was fully stressed in spite of the limited or narrow area of the compressive concrete. The steel network used in these beams was welded in all cases. In conclusion a seven point interpolation in the present Austrian specification is proposed. This interpolation includes recommendations for methods of testing to determine actual strength of beams produced for use.

Investigation of the coefficient of expansion of concrete (expansion joints in concrete highways)

A. Guttmann and K. Seidel, *Beton u. Eisen*, Vol. 24, p. 401, Dec. 20, 1936.

Reviewed by A. U. Theuer

The paper describes measurements of the coefficient of thermal expansion of concrete and then discusses, at considerable length, the significance of this constant in concrete highway design. The experimental work consisted of two series of tests. In the first nine concretes, similar in proportions, aggregate grading and workability but each representing a diffierent type of aggregate, were tested at the age of 65 days. These concretes contained 350 kgs. of cement per m³ of concrete, and sufficient water to yield an earth-damp to plastic mass. In a second series four concretes, similar to one of the above but each representing a different kind of cement, were tested at the age of two years. Specimens consisted of 10 by 10 by 30 cm. prisms. They were cured in their molds for one day, under damp cloth for six, and then in the air of the laboratory (20° C. 45 percent R. H.) until tested. All determinations were made in the range of 20 to 50° C. The apparatus consisted of an inner copper chamber surrounded by baffled water jackets through which water from a "thermostat" could circulate; together with a double-microscope comparator. Three repeated determinations were made on each specimen which was then dried in an oven at 100° C. for three hours followed by another determination. The values of the coefficient of expansion reported for the concrete, age 65 days, range from 15.6 to 10.9 x 10^{-6} per ° C. before drying and from 13.0 to 9.3 per ° C. after drying. For the concretes tested at the age of two years the values range from 13.5 to 11.8 per ° C. when tested as taken from the laboratory. It was found from measurements made soon after the oven drying treatment that the lengths of the specimens had increased. Measurements three months later, however, showed that the specimens returned practically to their original lengths. This increase in length, the authors conclude is a hysteresis effect, and from this conclusion they argue that the value of the coefficient of expansion obtained with increasing temperature, not the mean value obtained in a cycle, is the true value.

Shrinkage of concrete is discussed at length. Data from the work of four investigations of the relation between field and laboratory exposure of concretes are presented. It is concluded that concretes stored in the laboratory shrink considerably and over a long period, whereas concretes stored in the field (throughout Germany) show no progressive shrinkage but fluctuate in volume about a constant value. From this it is further concluded that, in practice, volume changes due to shrinkage are of minor importance compared to those due to temperature change and that a 12 mm. expansion joint in a 20 m. slab is sufficient.

Removing stains from cast stone and concrete

R. E. BAUMGARTEN, *Concrete Building and Concrete Products*, Vol. 12, No. 3, Mar. 1937, p. 50-52.
Reviewed by J. C. PEARSON

Author states that the following treatments have been tried and proved successful—*Iron Stains:* Mix 7 parts lime-free glycerine with a solution of 1 part sodium citrate in 6 parts lukewarm water, and make with it a thick paste of whiting or kieselguhr. Apply paste to stain with trowel, and scrape off when dried out. Repeat until stain has disappeared and wash thoroughly with clean water. Other suggestions are given for particularly bad cases.—*Tobacco Stains:* Dissolve 2 lbs. tri-sodium phosphate in 5 quarts of water. In separate vessel make a smooth stiff paste of 12 oz. of chloride of lime in water. Pour the former onto the paste and stir thoroughly. When the lime has settled, draw off the clear liquid and dilute with equal parts of water. Make a stiff paste of this with powdered talc and apply in the same way as described above for iron stains.—*Urine Stains:* Removed by same method as for tobacco stains.—*Smoke Stains:* Make smooth stiff paste of trichlorethylene and powdered talc and apply as described above. Cover with glass or pan to prevent rapid evaporation. If slight stain is left after several applications, wash thoroughly and then use the method described under tobacco stains. Precaution should be taken to ventilate a closed space in which trichlorethylene is used, as the fumes are harmful.—*Copper and Bronze Stains:* Mix 1 part ammonia with 10 parts water. Then thoroughly mix 1 lb. of powdered talc and 4 oz. of dry ammonia chloride. Make a smooth stiff paste of these mixtures and spread over stain at least $\frac{3}{8}$" thick. Allow paste to dry out, scrape off and wash with clean water. Repeat if necessary. *Ink Stains:* Dissolve $\frac{1}{2}$ lb. chloride of lime in 2 quarts water. Allow to stand until the lime has settled. Filter the liquid and add to it $\frac{1}{2}$ oz. of 24% acetic acid. Soak a piece of flannel in this, place it on the stain, and cover with a piece of glass. If stain has not disappeared when paste has dried out, repeat the application.—*Mineral Oil Stains:* Same treatment as for smoke stains.—*Stains from Linseed Oil, Palm Oil or Sand·* Proceed as described under smoke stains. If, after repeated applications, the stain is still visible, apply the amomnia paste described under copper and bronze stains. If, finally any trace of the stain is left, use the following method. Mix in dry state, 2 oz. tri-sodium phosphate, 3 oz. sodium perborate, and 6 oz. powdered talc. Dissolve 15 oz. soft soap in $4\frac{1}{2}$ pints hot water, pour this onto the dry mix and stir thoroughly. Trowel some of the paste onto the stain, leave until dry, and carefully remove it. Soak a piece of flannel in equal parts of acetone and amylacetate and place over the stain. Cover with glass or pan to prevent rapid evaporation. The procedure may be repeated if necessary, always thoroughly drying the surface before repeating application.

Strength tests for cement

W. H. GLANVILLE, *The Structural Engineer*, (London, England) Feb., 1937, p. 50-63.
HIGHWAY RESEARCH ABSTRACT

Tests were made to determine the influence of amplitude and frequency of vibration on the crushing strength of 1:3 mortar and $12\frac{1}{2}$ per cent of water, the duration of the vibration being 2 minutes. From the results tabulated it will be observed that with one exception the crushing strength with a given amplitude rises with the speed, but the increase is small within the range investigated, and all tests with a given cement give roughly the same crushing strength. From these results it can be concluded that the crushing strength of portland or high alumina cement mortar mixed with 12.5 percent of water is practically independent of the amplitude and

frequency for vibrations having accelerations between about 5 and 10 g. and frequencies between 2,500 and 13,000 vibrations per minute

INFLUENCE OF AMPLITUDE AND FREQUENCY OF VIBRATION

Amplitude (Displacement from Mid-Position Inches)	Acceleration (Gravity)	Frequency	1:3 Mortar, 12½ Per Cent Water			
			Normal Portland Cement		High Alumina Cement	
		Vibrations per Minute	Compressive Strength at 7 Days p.s.i.*	Range—Per Cent**	Compressive Strength at 7 Days p.s.i.*	Range—Per Cent**
.0335	6 g.	2,510	2,826	6.7	7,833	6.6
	10 g.	3,240	2,919	11.5	8,227	10.9
.0134	6 g.	3,980	2,873	6.6	7,915	9.9
	10 g.	5,130	2,845	11.3	8,083	4.5
.00568	4 g.	5,000	2,771	4.7	7,213	20.3
	9½ g.	7,640	2,954	5.1	8,032	5.9
.00214	2½ g.	6,420	3,023	8.4	7,962	6.9
	10 g.	12,820	3,075	6.2	8,382	6.6

*Mean of 6 tests.

**Per cent range = $\frac{\text{(max. — min.)}}{\text{mean}}$ x 100.

An investigation was made to determine the lower limit of the water content at which the cubes could be properly consolidated by vibration. One series of cubes was compacted at 12,800 vibrations per minute and a second series at 3,240 vibrations per minute, the accelerations in both cases being 10 g. The approximate limit of dryness for vibration compacting at 12,800 vibrations per minute and 10 g. (amplitude = .00214 in.) was about 8¾ per cent for high alumina cement and about 9¼ percent for portland cement, the limits being judged merely from the appearance and weight of the vibrated cubes. At a frequency of 3,240 vibrations per minute, however, and an acceleration of 10 g. (amplitude = .0335 in.) it was found that the water content could not be reduced below about 11 percent for portland cement and below about 10 percent for high alumina cement. With dry mixes at large amplitudes and medium and low frequencies it was found that the mix tended to revolve as a whole about a horizontal axis and failed to compact.

Reconditioning deteriorated portland cement

Concrete, Mar., 1937, p. 137. HIGHWAY RESEARCH ABSTRACTS

The reconditioning of deteriorated cement, through the addition of a weak solution of hydrochloric acid, is covered by B. G. Skramtajev of the Central Institute for Building Research, Moscow, in the Jan. 1937 issue of *Cement and Lime Manufacture* (London).

Factors responsible for the deterioration of cement stored for long periods are well known. Primarily the moisture contained in the air affects the cement in the same way as the mixing water, hydrate and hydrosilicate of lime being slowly formed on the surface of the particles. The cement particles are also subjected to the action of carbon dioxide in the air, which changes the calcium hydroxide into calcium carbonate and the hydrosilicate of lime into calcium carbonate and amorphous free silicic acid. As a result, the particles become coated with an insoluble complex film of these two substances. This film prevents combination of the cement with mixing water, especially when the mixing is gentle and no vigorous mechanical stress is exerted upon the film.

It was proposed by Professor Skramtajev to improve cement deteriorated after storage by adding a weak solution of hydrochloric acid to the mixing water, with the object of destroying the carbonate content of the films on the cement particles in accordance with the formula:

$$CaCO_3 + 2\,HCl = CaCl_2 + CO_2 + H_2O$$

This addition allows the water to reach the unaltered cement particles and thus restores the quality of the cement, while calcium chloride accelerates the hardening. Calcium chloride by itself does not hasten the hardening of cements which have been deteriorated due to prolonged storage, as it is unable to destroy the carbonate films.

Cement was artificially "aged" by passing a current of carbonic acid and water vapor through it; it was then mixed with a 4 per cent solution of hydrochloric acid. A stronger solution might have been used if this were possible with the setting time of the cement. Results of these efforts to improved spoiled cement are shown in the table:

REDUCTION IN STRENGTH DUE TO ARTIFICIAL AGING

	Compressive Strength; kg. per sq. cm. of 1:3 Mortar		
	4 Days	7 Days	28 Days
Fresh cement	163	159	273
"Aged" cement	62	105	138

STRENGTH AFTER ADDITION OF HYDROCHLORIC ACID

"Aged" cement	119	178	227

The great fire in the Venlo (Holland) Market building

G. A. DE BOER, *Beton u. Eisen*, Vol. 36, No. 7, p. 113, Apr. 5, 1937. Reviewed by A. U. THEUER

This article describes the results of an investigation and the pertinent factors involved in the complete collapse of a reinforced concrete market building in Venlo, Holland. The history of this structure is stated to completely verify the facts gathered at the site of the wreckage after the fire. The building was a two-story 50 by 21 m reinforced concrete structure faced with masonry. A complete reinforced concrete skeleton was intended; however, in the actual construction the concrete frame was not carried up to the roof-ridge. The roof rested on a masonry parapet. There was no expansion joints anywhere. The plans for the building were drawn by the contractor, an individual who in this case had had no previous experience with reinforced concrete construction. There had been no inspection during construction by either the government or the owners. The concrete was very poor, consisting of a 1:2:3 mix. The aggregates were unwashed pit-run sand and gravel. The fire which started at 11:00 p. m. was not brought under control until 3:00 p. m. the following day. In the course of the fire, cracks first appeared along the roof ridge, followed by a shearing off of the masonry parapet. A little later the roof collapsed over the center. A 30 m length of the building was completely destroyed, the remaining 20 m were at the point of failure. The investigation revealed the following facts. The construction was not monolithic. The roof girders were not tied into the masonry walls, even though they were treated as statically indeterminate members in the design. The beams were not continuous over the intermediate supports. In the course of the investigation it was found that neither the stirrups nor longitudinal steel had been carried up into the flanges of T-beams. In the part of the building remaining after the fire flanges and webs were found to have

separated completely in these beams, much of the concrete below the tension steel having spalled off. Samples of the concrete investigated in a laboratory revealed a very weak material with faulty aggregate grading. Both the plans and the stress computations were found to contain outright errors which could be traced in the structure itself. The following analysis of the failure is given. Built up against another building at one end, deformations taking place in the structure were forced in the opposite direction; the amount depending on the length of the building, its temperature, and the coefficient of expansion of the surrounding masonry. Since the coefficient of expansion of the masonry is less than that of concrete and since the thin roof slab would heat up more rapidly than the heavier walls and transverse beams, a shearing force was produced between the web and flanges of the T-beams that was larger than the shearing strength of the concrete. The flanges of the T-beams separated from the webs which then became rectangular beams of small depth. Becoming overloaded these beams sagged badly causing a spalling of concrete under the tension reinforcement, which after exposure heated quickly causing failure. Computations based on the laboratory determined strength of the concrete bear out the conclusion that the shearing force in the T-beams was greater than the shearing strength of the concrete. This is further born out by the findings in that part of the building still remaining. The shearing away of the parapet proves the existence of a large lateral force, caused by the expansion of the slab after its separation from the beams (webs). The collapse of the roof a little later bears out the second part of the theory. The conclusion drawn, is that a reinforced concrete building, improperly designed and constructed, can be completely destroyed by fire.

Concrete pavement crack survey

E. W. Bauman, (Engineer of Materials and Tests, Tennessee Department of Highways and Public Works, Nashville, Tennessee) July, 1936. Highway Research Abstracts

During the past several years pavements in Tennessee have been constructed with dummy joints spaced at intervals of 30, 40 and 50 ft., and more recently with steel dowels at dummy joints. A survey was made of the surface conditions of these pavements with the object of obtaining data regarding the value of dowels and the proper spacing of dummy joints for a satisfactory slab design. Observations were made on five sections totaling 109 miles. Description of the design of these sections and the crack data obtained are as follows:

A. 24.98 miles of 8-in. 6-in. 8-in. cross section, having dummy joints spaced at 30-ft. intervals; dowels at dummy joint ; ¾-in. expansion joint spacing every 90 ft.

B. 6.84 miles of 8-in. 6-in. 8-in. cross section, with dummy joints at 30-ft. intervals without dowels; ¾-in. expansion joint every 90 ft.

C. 28.5 miles of 9-in. 7-in. 9 in. cross section, with dummy joints at 40-ft. intervals, without dowels; 2½-in. expansion joint every 320 ft.

D. 25.5 miles of 9-in. 7-in. 9-in. cross section, with dummy joints at 50-ft. intervals, without dowels; 2½-in. expansion joint every 300 ft.; also 6.35 miles of 8 in. 6-in. 8-in. cross section under this same group.

E. 17.05 miles of 8-in. 6-in. 8-in. cross section, without dummy joints; 2½-in. expansion joint every 500 ft.

All coarse aggregates used were Tennessee River Gravel, classified as siliceous, cherty gravel. The sand is river sand produced from the same bars from which the gravel was obtained. The concrete was designed for a strength of 750 lb. modulus

of rupture at 28 days. It is thought that sufficient mileage was inspected to eliminate in large measure the effect of variable subgrade conditions.

SURVEY DATA

Section	Corner Breaks		Cracks*	Average Slab Length** Feet
	Outside	Center		
	Average Number Per Mile			
A	1.0	0.16	181.3	29.1
B	0.15	0.0	186.2	28.4
C	0.11	0.0	146.0	36.2
D	0.75	7.53	167.5	31.5
E	0.94	1.99	219.7	24.0

*Average number of cracks per mile includes dummy joints, expansion joints and cracks developed subsequent to construction of pavement.

**Average slab length is obtained by dividing total length of pavement in feet by the number of all cracks, and indicates the sections of unbroken concrete.

Undoubtedly the dowels provided for load transfer from one section of pavement to another, but it is difficult to place dowels properly, and the damage caused by improper installation outweighed the benefits derived from their function as load transfer units. In one instance transverse cracks developed midway between the dummy joints and were not permitted to form at the joints because the dowels functioned improperly.

As regards corner breaks, it is notable that more such breaks occurred in the doweled dummy joint section A than in section B pavements without dowels.

From the standpoint of maintenance cost, appearance and riding quality, the pavement with the least number of cracks is the most desirable. The fact that dummy cracks contribute a definite control over ultimate condition of the pavement is shown by comparing section E with any of the other four. Forty ft. appears to be the proper spacing of dummy joints for the conditions encountered in these sections.

Vibration in concrete pavement construction

J. E. BOYD (Testing Engineer, Georgia State Highway Department) May, 1936
HIGHWAY RESEARCH ABSTRACTS

During the construction of State projects in May, 1935, the contractor was authorized to try out the Blaw-Knox pan type vibrator as an auxiliary unit of the finishing machine. The semi-flat pan vibrating member, 20 ft. long, was suspended from the finisher chassis, and driven by a separate four-cylinder gasoline engine. The vibrating member could be raised or lowered as desired.

The normal mix required 1.45 bbl. of cement per cu. yd. and a maximum of 6.2 gal. of water per bag of cement producing a slump of approximately 2 inches. With the vibrator a sufficient quantity of coarse aggregate was added to reduce the cement factor to 1.36 and the slump to about 1 in. This mix was easily spread with the vibrator operating at frequencies between 3600 and 4000 r.p.m., but because too much water was being wasted over the forms, the aggregate was increased to bring the slump to about ½ in. and the cement factor to 1.29. This mix was spread without apparent difficulty at slightly increased vibrating frequency. The waste of water and mortar over forms was also eliminated.

On a second project a Blaw-Knox gas electric pan type vibrator was used, having two 10-ft. pan vibrating members with a 3 h.p. electric motor. The normal mix for this project, using gravel, was 1.50 bbl. of cement per cu. yd. of concrete, and not

more than 5¾ gal. of water per bag of cement. Sufficient aggregate (mostly gravel coarse aggregate) was added to the normal mix to diminish the cement factor from 1.50 to 1.35 and the slump from 2 in. to ¾ in. This mixture was very easily spread and finished with the vibrator operating at the minimum frequency of approximately 3600 r.p.m.

In most discussions concerning the effect of vibration upon density, comparison has been made between the specific gravity of the vibrated concrete core and that of the non-vibrated. While this may fairly well indicate the general effect of vibration, some or all of the effect may be due to changes in composition. The theoretical specific gravity of the vibrated concrete is necessarily higher (.03 to .04 in this study) than the theoretical specific gravity of non-vibrated concrete due to the lower volume percentage of water used. The effect of vibration should, therefore, be expressed by comparing the ratios of actual specific gravity to theoretical, or by comparing the ratios of actual voids to theoretical voids.

In the first mentioned project, the calculated percentage of voids for the non-vibrated mixture was 13.5 and the actual voids averaged 12.10 percent. The ratio of actual voids to calculated voids is .8965 percent.

The calculated voids for the vibrated mixture were 12.22 per cent, and the actual average voids 9.60 per cent. The ratio was .7855.

The ratio of voids in vibrated concrete (9.6) to voids in non-vibrated concrete (12.10) was .794.

The ratio of actual voids in vibrated concrete (9.6) to calculated voids in non-vibrated concrete (13.5) was .711.

In the second project the calculated percentage of voids for the non-vibrated mixture was 12.61 and the actual voids averaged 10.55 per cent. Thus the ratio of actual voids in non-vibrated concrete to calculated voids in the non-vibrated was .828.

The calculated voids in the vibrated mixture averaged 11.42 per cent and the actual average voids 10.7 per cent. Thus the ratio of actual average voids in vibrated concrete to calculated voids in the vibrated mix was .937.

The ratio of voids in vibrated concrete (10.70) to voids in non-vibrated concrete (10.44) was 1.025.

The ratio of actual voids in vibrated concrete (10.70) to calculated voids in normal non-vibrated concrete (12.61) was .8485.

From this study of densities it was concluded that the amplitude of vibration employed in this second project was inadequate. For mixtures with slumps below one inch, the frequency of vibration for the Blaw-Knox vibrator obviously should exceed 3600 r.p.m.

Regarding the future of plain and reinforced concrete—concrete problems

Beton u. Eisen, Nos. 1 and 2, Vol. XXXVI, p. 1, Jan. 5 and 20, 1937. Reviewed by A. U. Theuer

Under the heading "Future Problems in Plain and Reinforced Concrete" the January 5 issue of *Beton u. Eisen* is given over almost entirely to discussions representing the viewpoints or ideas of prominent individuals on just how matters in the concrete field might be improved. Suggestions regarding conditions, relations, or specifications that will aid or obstruct progress, ideas regarding developments, and ways or means to circumvent present-day limitations in concrete construction are being invited and requested by the editors, in the form of terse discussions. These

will be published from time to time in future issues of *B. u E.* under this general heading of "Future Problems."

The first six discussions deal with specifications, government inspection, and owner-engineer-contractor relations. They are of interest, perhaps, principally in that their problems in building relations are not entirely dissimilar from those existing in the United States. J. Luhrs criticizes clauses in the German specifications differentiating allowable working stresses permitted "competent" and "very competent" designers, on the basis of ambiguity, contending that the only defensible distinction would be between competent and incompetent designers. Moreover this and other ambiguous terminology has in practice prevented the use of high design stresses rather than eliminating incompetent designers from the field, considered to be a real need. Ulrich Fischer ascribes as one of the causes of unsatisfactory conditions in contract letting the free design service given by certain building firms. In practice, he states, this generally turns to the advantage of those firms having inadequate engineering staffs. The saving effected in this way permits underbidding. General knowledge is called upon too often where specific knowledge of special technical problems is required. H. Craemer complains of insufficient time permitted the builder in planning his schedule and successive stages in construction. Bela Enyedi describes the generally satisfactory conditions obtained in Hungary in the letting of contracts for engineering structures. Alfred Millies would favor in concrete construction a group of special engineer-contractors recommended by a sound education and years of active practice in concrete work by permitting them a wider latitude in the use of high working stresses. This, he states, has proved to be a highly successfuly device in steel construction. A second recommendation is to make aggregate approval tests rest on actual cylinder strengths, the tests to be made by the producer. Dr. Thullie (Poland) defends an all inclusive building code and stringent inspection rather than legal reprisal after failures in practice. H. Kayser lists the following problems as pressing for immediate solution; (1) stresses in concrete elements immediately after set and at early ages; (2) significance of plastic flow in actual structures; (3) conclusions from which practical recommendations may be drawn regarding effect of repeated loading wherever this is of significance; (light structure subjected to heavy moving loads); (4) a means whereby optical methods may be applied to three dimensional stress problems in concrete; and (5) a complete study of welding in concrete reinforcement. M. Spindel (Austria) traces the successive steps in the development of concrete and cements. Due to the misapplication of theoretical knowledge and fundamentals he notes certain regressive steps creeping in every time a new development of principle is set forward. Thus in the development of high-early-strength cements, concentration on this one property to the neglect of equally important properties, led to the use of too high lime contents, hidden retarders, excessively fine grinding, etc. until failures in practice occurred. This in turn gave an impetus to the development of a vast variety of special cements, considered by the author, a second step in reverse. A number of equally important points are also discussed. F. Baravalle in a complete paper carries through computations and by graphs illustrates his conclusions regarding the maximum attainable spans for reinforced concrete arches of varying rises. He assumes as working stresses for concrete 100 and 150 kgs/cm^2. The maximum spans considered range from 40 to 1220 m. In an appended discussion he considers means whereby these spans might be further increased. Erich Friederich contributes an interesting method of dimensioning concrete sections, heavily reinforced. This method takes into consideration plastic properties in concrete. Lightly reinforced sections would be

designed by the conventional triangular stressed concrete area. For the dividing line between design methods the author refers the reader to a previous paper.

Under the heading of Future Problems (Jan. 20), A. Hawranek discusses the question of long span reinforced concrete arches with particular regard to plasticity. In his discussion of plasticity the author is quite inclusive in pointing out the possible effects of different theories regarding this property on concrete arch design. A short description of the two theories advanced to explain plastic flow are sketched. A discussion of desirable research and specific problems pertinent to arch design awaiting experimental study are pointed out at length. In this same issue the firm of Beton u. Tiefbau-Mast criticizes the practice of aggregate separation and recombination when carried to an extreme, a present tendency (Germany). The separation of the particles smaller than 3 mm. requires washing as an aid and the loss of a large amount of the fines is stated to be undesirable.

INDEX

PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

VOLUME 33—1937

From JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Vol. 8, Sept.-Oct., 1936, to May-June 1937

This is an Index of:

Original contributions to the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE—papers, reports, discussions by subject, title and author.

For the convenience of the reader who is referring to JOURNAL issues rather than to Bound Volumes (in which discussion is assembled following the paper discussed) there is a reference to JOURNAL issue in which discussion appeared. This reference is by supplementary page numbers. If the last page of a paper is 16, the first page of the discussion, published later, is 16-1.

In general, important subjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN. "Surface treatment" and other subjects classified under this head, are indented.

Key words to important subjects, appear, in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Blast furnace slag," each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names and original titles of papers appear in bold face type in proper alphabetical sequence with the subjects, with references to their contributions.

Specific data on Beams are so indexed by reference to ENGINEERING DESIGN or TESTS OF MEMBERS AND COMPLETED STRUCTURES thus avoiding an oversight by the searcher of important allied data.

The readiest use may be made of this index by gaining some familiarity with the main classifications as follows:

ARCHITECTURAL DESIGN

BIBLIOGRAPHIES

BREAKWATERS—DOCKS AND SEA WALLS

BRIDGES

BUILDING CODES

BUILDINGS

BUILDING UNITS

CENTRAL MIXING

COMMITTEES

CONSTRUCTION—DETAILS, METHODS AND COSTS

CURING

DAMS

DURABILITY

ENGINEERING DESIGN

FAILURES

FIRES, FIRE TESTS AND FIRE-PROOFING

LAWS AND PROPERTIES

MASONRY UNITS

MATERIALS AND TESTS

MIXTURES

PAVEMENT

PERMEABILITY

PIPE

PRECAST UNITS

RAILWAY USES

RESEARCH

RESERVOIRS AND TANKS

SEWERS

SPECIFICATIONS

STRUCTURAL MEMBERS

SURFACES

TESTS AND FIELD CONTROL

TESTS OF MEMBERS AND COMPLETED STRUCTURES

VIBRATION

WATER WORKS

A

Admixtures—See MATERIALS AND TESTS
Ahlers, John G.—Experiences of an American contractor in London, England..... 613
Anderson, Arthur R.—A study of sub-aqueous concrete..... 339
—Disc. (May-June '37 Jl.).....346–1
Andrew, Charles E.—Disc. Recent developments in foundation design with special reference to concrete. (Suppl. Sept.-Oct. '37 Jl.).....556–1

ARCHITECTURAL DESIGN

Buildings—Structures in Europe. Boase... 521
—Disc. Moreell. (Suppl. Sept.-Oct. '37 Jl.)..... 540–1
Aseismic design for buildings. Creskoff... 223

B

Bates, P. H.—Disc. Portland-puzzolan cement as used in the Bonneville spillway dam. (May-June '37 Jl.).....222–1

BIBLIOGRAPHIES

Design—For flexure and compression. Whitney..... 483
—Disc. Johnston, Lyse, Mensch, Morris, Urquhart, Wernisch, Whitney. (Suppl. Sept.-Oct. '37 Jl.).....498–1
Earthquake action. Creskoff..... 223
Portland cement—Conductometric analysis. Boast..... 131
Properties — Job-cured specimens — Committee 107 report. Edwards..... 62
—Disc. Edwards, Scofield. (Jan.-Feb. '37 Jl.)..... 64–1
Bleeding test—Basis for evaluating fines. Sprague..... 20
—Disc. Scripture, Sprague. (Jan.-Feb. '37 Jl.)..... 40–1
Boase, A. J.—Notes on inspection of structures in Europe..... 521
Boast, W. B.—A conductometric analysis of portland cement pastes and mortars and some of its applications..... 131
Bonneville spillway dam. Clark and Brown..... 183
—Disc. Bates, Carlson, Colburn, Clark and Brown, Davis. (May-June '37 Jl.).....222–1

BRIDGES

Foundation — Design — Recent developments. Proctor..... 541
—Disc. Andrew, Proctor. (Suppl. Sept.-Oct. '37 Jl.).....556–1
Railroad — Maintenance and repair. Hirschthal..... 251
Rigid frame—Rapid and long-time tests of. Richart and Olson..... 459
—Disc. Hadley, Richart and Olson, Whitney. (Suppl. Sept.-Oct. '37 Jl.).....482–1
Brown, Jr., H. E.—Portland-puzzolan cement as used in the Bonneville spillway dam..... 183
—Disc. (May-June '37 Jl.).....222–1

BUILDING CODES

Columns—A. C. I. regulations. Willson... 311
—Disc. Richart, Willson. (May-June '37 Jl.).....318–1
Standard—Committee 501 report. Stephens and Zipprodt..... 503

BUILDINGS

Factory — Construction in England. Ahlers..... 613
Foundation — Design — Recent developments. Proctor..... 541
—Disc. Andrew, Proctor. (Suppl. Sept.-Oct. '37 Jl.).....556–1
General—Earthquake resistant. Creskoff. 223
General—European. Boase..... 521
—Disc. Moreell. (Suppl. Sept.-Oct. '37 Jl.).....540–1

C

Carlson R. W.—Disc. Portland-puzzolan cement as used in the Bonneville spillway dam. (May-June '37 Jl.)...222–1
—Drying shrinkage of large concrete members..... 327
—Properties of cements and concretes containing fly ash..... 577
Casey, Hugh J.—Cement and concrete studies on the Passamaquoddy tidal power project..... 279
Cement and concrete studies on the Passamaquoddy tidal power project. Casey..... 279
Clark, R. R.—Portland-puzzolan cement as used in the Bonneville spillway dam..... 183
—Disc. (May-June '37 Jl.).....222–1
Colburn, D. S.—Disc. Portland-puzzolan cement as used in the Bonneville spillway dam. (May-June '37 Jl.)...222–1

COMMITTEES

Committee 107 report—Properties of job-cured concrete at early ages. Edwards. 41
—Disc. Edwards, Scofield. (Jan.-Feb. '37 Jl.)..... 64–1
Committee 313 report—Effect of plastic flow and volume changes on design. Morris..... 123
—Disc. Douglass, Richart, Shank, Whitney. (May-June '37 Jl.)....130–1
Committee 501 report—Standard building code—Building Regulations for reinforced concrete. Stephens and Zipprodt 503

Committee 710 report—High early strength cement in concrete masonry manufacture. Wilk 499
Compacting factor—Relation to workability 319
Concrete: Its maintenance and repair. Young 367
Concrete rehabilitation work on the Uncompahgre project. Reeves 303
Conductometric analysis of portland cement pastes and mortars and some of its applications. Boast 131

CONSTRUCTION—DETAILS, METHODS AND COSTS

Bridge—Structures in Europe. Boase 521
—Disc. Moreell. (Suppl. Sept.-Oct. '37 Jl.) 540–1
Building — Foundation — Structures in Europe. Boase 521
—Disc. Moreell. (Suppl. Sept.-Oct. '37 Jl.) 540–1
Building — In England — By American contractor. Ahlers 613
Dam—Bonneville spillway—Use of portland-puzzolan cement. Clark and Brown 183
—Disc. Bates, Carlson, Colburn, Clark and Brown, Davis. (May-June '37 Jl.) 222–1
Foundations—Pile—Recent developments. Upson 557
General—European practice. McMillan 395
Maintenance and repair. Young 367
Pavement—Vibration—Effect on strength. Jackson and Kellermann 411
Railroad structure — Maintenance and repair. Hirschthal 251
Sub-aqueous structures. Anderson 339
—Disc. Stanton, Anderson. (May-June '37 Jl.) 346–1
Tremie concrete. Anderson 339
—Disc. Stanton, Anderson. (May-June '37 Jl.) 346–1
Tunnel lining — Uncompahgre project. Reeves 303
Corrosion—See DURABILITY.
Creskoff, Jacob J.—Earthquakes and reinforced concrete 223

CURING

Job conditions—Committee 107 report. Edwards 41
—Disc. Edwards, Scofield. (Jan.-Feb. '37 Jl.) 64–1
Salt water — Passamaquoddy project. Casey 279

D

DAMS

Gravity—Bonneville spillway. Clark and Brown 183
—Disc. Bates, Carlson, Colburn, Clark and Brown, Davis. (May-June '37 Jl.) 222–1
Davis, Harmer E.—Properties of cements and concretes containing fly ash 577
Davis, Raymond E.—Disc. Portland-puzzolan cement as used in the Bonneville spillway dam (May-June '37 Jl.) 222–1
—Properties of cements and concretes containing fly ash 577
Derleth, C. P.—Disc. Properties of cements and concretes containing fly ash. (Suppl. Sept.-Oct. '37 Jl.) 612–1
Design of reinforced concrete members under flexure or combined flexure and direct compressions. Whitney 483
—Disc. Johnson, Lyse, Mensch, Morris, Urquhart, Wernisch, Whitney. (Suppl. Sept.-Oct. '37 Jl.) 498–1
Douglass, A. S.—Disc. Effect of plastic flow and volume changes on design—Report of Committee 313. (May-June '37 Jl.) 130–1
Drying shrinkage of large concrete members. Carlson 327

DURABILITY

Corrosion—Effect of sodium sulphate. Tuthill 83
—Disc. Kinzie, Tuthill. (Mar.-Apr. '37 Jl.) 108–1
Disintegration—Maintenance and repair. Young 367
Disintegration — Tunnel lining — Uncompahgre project. Reeves 303
Maintenance and repair—railroad structures. Hirschthal 251

E

Earthquakes and reinforced concrete. Creskoff 223
Edwards, Harlan H.—Committee 107 report—Properties of job-cured concrete at early ages 41
Effect of calcium and sodium chlorides on concrete when used for ice removal. Gonnerman, Timms and Taylor 107
Effect of vibration on the strength and uniformity of concrete pavement. Jackson and Kellermann 411

ENGINEERING DESIGN

Bridge—Structures in Europe. Boase 521
—Disc. Moreell. (Suppl. Sept.-Oct. '37 Jl.) 540–1
Buildings—Earthquake resistant. Creskoff 223
Building — Foundation — Structures in Europe. Boase 521
—Disc. Moreell. (Suppl. Sept.-Oct. '37 Jl.) 540–1

—In England—By American contractor. Ahlers 613
Columns—A. C. I. regulations. Willson .. 311
—Disc. Richart, Willson. (May-June '37 Jl.) 318–1
Foundations—Pile—Recent developments. Upson 557
—Recent developments. Proctor 541
—Disc. Andrew, Proctor. (Suppl. Sept.-Oct. '37 Jl.) 556–1
Frames—Continuous—Moments in. Hollister 147
—Disc. Floris. (Mar.-Apr. '37 Jl.) 170–1
—Rigid knee. Richart and Olson 459
—Disc. Hadley, Richart and Olson, Whitney. (Suppl. Sept.-Oct. '37 Jl.) 482–1
General—European practice. McMillan .. 395
Members — Flexure and compression. Whitney 483
—Disc. Johnston, Lyse, Mensch, Morris, Urquhart, Wernisch, Whitney. (Suppl. Sept.-Oct. '37 Jl.) 498–1
Plastic flow and volume changes—Effect on design. Morris 123
—Disc. Douglass, Richart, Shank, Whitney. (May-June '37 Jl.) 130–1
Rigid frames—Moments in. Hollister 147
—Disc. Floris. (Mar.-Apr. '37 Jl.) 170–1
Slabs—Reinforcement. Lyse and Wernisch 1
—Disc. Ernst, Hajnal-Konyi. (Jan.-Feb. '37 Jl.) 16–1
Ernst, G. C.—Disc. A study of reinforcement in concrete slabs. (Jan.-Feb. '37 Jl.) 16–1
European concrete practice. McMillan ... 395
Evaluating fines in concrete on a bleeding test basis. Sprague 29
—Disc. Scripture, Sprague. (Jan.-Feb. '37 Jl.) 40–1
Experiences of an American contractor in London, England. Ahlers 613

F

FAILURES
Canal lining — Uncompahgre project. Reeves 303
Floors—Metallic aggregate in concrete floors. Scripture 17
—Disc. Roelofsen. (Jan.-Feb. '37 Jl.) 28–1
Floris, A.—Disc. A short method for computing moments in continuous frames. (Mar.-Apr. '37 Jl.) 170–1

G

Glanville, W. H.—Grading and workability. 319
Gonnerman, H. F.—Effect of calcium and sodium chlorides on concrete when used for ice removal 107
Grading and workability. Glanville 319

H

Hadley, Homer M.—Disc. Rapid and long-time tests on reinforced concrete knee-frames. (Suppl. Sept.-Oct. '37 Jl.) 482–1
Hajnal-Konyi, K.—Disc. A study of reinforcement in concrete slabs. (Jan.-Feb. '37 Jl.) 16–1
Hirschthal, M.—Maintenance and repair of concrete structures in railroad construction 251
Hollister, S. C.—A short method for computing moments in continuous frames 147
—Disc. (Mar.-Apr. '37 Jl.) 170–1

I

Ice removal—Effect of calcium and sodium chlorides. Gonnerman, Timms and Taylor 107
Institute activities.—Review of. McMillan 363
Institute carries on. McMillan 363

J

Jackson, F. H.—The effect of vibration on the strength and uniformity of concrete pavement 411
Johnston, Bruce—Disc. Design of reinforced concrete members under flexure or combined flexure and direct compression (Suppl. Sept.-Oct. '37 Jl.) 498–1

K

Kellermann, W. F.—The effect of vibration on the strength and uniformity of concrete pavement 411
Kelly, J. W.—Properties of cements and concretes containing fly ash 577
Kinzie, Jr., Robert A.—Disc. Resistance of cement to corrosive action of sodium sulphate solutions. (Mar.-Apr. '37 Jl.) 106–1

L

LAWS AND PROPERTIES
Abrasion—Resistance to—Metallic aggregate. Scripture 17
—Disc. Roelofsen. (Jan.-Feb. '37 Jl.) 28–1
Durability—Corrosive action of sodium sulphate. Tuthill 83
—Disc. Kinzie, Tuthill. (Mar.-Apr. '37 Jl.) 106–1
—Disintegration — Maintenance and repair. Young 367
Electric conductivity—Pastes and mortars. Boast 131
General—Effect of fly ash. Davis, Carlson, Kelly and Davis 577
—Disc. Derleth. (Suppl. Sept.-Oct. '37 Jl.) 612–1

—Passamaquoddy project studies. Casey. 279
Physical—Job-cured specimens—Committee 107 report. Edwards. 41
—Disc. Edwards, Scofield. (Jan.-Feb. '37 Jl.) 64–1
Plastic flow and volume changes—Effect on design. Morris. 123
—Disc. Douglass, Richart, Shank, Whitney. (May-June '37 Jl.) 130–1
Volume changes and plastic flow—Effect on design. Morris. 123
—Disc. Douglass, Richart, Shank, Whitney. (May-June '37 Jl.) 130–1
—Shrinkage — Drying large members. Carlson. 327
Workability—Effect of grading of aggregate. Glanville. 319
Lyse, Inge—A study of reinforcement in concrete slabs. 1
—Disc. Design of reinforced concrete members under flexure or combined flexure and direct compression. (Suppl. Sept.-Oct. '37 Jl.) 498–1

M

Maintenance and repair of concrete structures in railroad construction. Hirschthal. 251

MASONRY UNITS

Manufacture—Use of high early strength. Wilk. 499

MATERIALS AND TESTS

Admixtures—Fly ash. Davis, Carlson, Kelly and Davis. 577
—Disc. Derleth. (Suppl. Sept.-Oct. '37 Jl.) 612–1
Admixtures—In maintenance and repair. Young. 367
—Evaluating by bleeding test. Sprague. 29
—Disc. Scripture, Sprague. (Jan.-Feb. '37 Jl.) 40–1
Aggregates—Grading—Effect on workability. Glanville. 319
—Metallic. Scripture. 17
—Disc. Roelofsen. (Jan.-Feb. '37 Jl.) 28–1
—Passamaquoddy project. Casey. 279
Calcium chloride—Effect on concrete in ice removal. Gonnerman, Timms and Taylor. 107
Cements—Effect of fly ash. Davis, Carlson, Kelly and Davis. 577
—Disc. Derleth. (Suppl. Seppt.-Oct. '37 Jl.) 612–1
—High early strength—Masonry manufacture. Wilk. 499
—Passamaquoddy project. Casey. 279
—Portland-puzzolan — Bonneville dam. Clark and Brown. 183
—Disc. Bates, Carlson, Colburn, Clark and Brown, Davis. (May-June '27 Jl.) 222–1
—Resistance to sodium sulphate. Tuthill. 83
—Disc. Kinzie, Tuthill. (Mar.-Apr. '37 Jl.) 106–1
Paint—Maintenance. Young. 367
Portland cement—Conductometric analysis. Boast. 131
—Specifications—Federal. Moreell. 435
Sodium chloride—Effect on concrete in ice removal. Gonnerman, Timms and Taylor. 107
Sodium sulphate—corrosive action. Tuthill. 83
—Disc. Kinzie, Tuthill. (Mar.-Apr. '37 Jl.) 106–1
Waterproofing — In maintenance work. Young. 367
McMillan, F. R.—The Institute carries on. . . 363
—Some comparisons of European and American concrete practice. 395
Mensch. L. J.—Disc. Design of reinforced concrete members under flexure or cobmined flexure and direct compression (Sept.-Oct. '37 Jl.) 498–1
Metallic aggregate in concrete floors. Scripture. 17
—Disc. Roelofsen. (Jan.-Feb. '37 Jl.) 28–1
Mississippi river bridge—Foundation design. Proctor. 541
—Disc. Andrew, Proctor. (Suppl. Sept.-Oct. '37 Jl.) 556–1
Moisture diffusion constant — Drying shrinkage. Carlson. 327
Moreell, Ben—Disc. Notes on inspection of structures in Europe. (Suppl. Sept.-Oct. '37 Jl.) 540–1
—The new Federal specifications for portland cements. 435
Morris, Clyde T.—Disc. Design of reinforced concrete members under flexure or combined flexure and direct compression. (Suppl. Sept.-Oct. '37 Jl.) 498–1
—Report of committee 313—Effect of plastic flow and volume changes on design. 123

N

New Federal specifications for portland cements. Moreell. 435
Notes on inspection of structures in Europe. Boase. 521
—Disc. Moreell. (Suppl. Sept.-Oct. '37 Jl.) 540–1

O

Olson, T. A.—Rapid and long time tests on reinforced concrete knee frames. 459
—Disc. (Suppl. Sept.-Oct. '37 Jl.) . . 482–1

P

Painting—In maintenance. Young..... 367
Passamaquoddy tidal project. Casey..... 279
Pavement—Effect of calcium and sodium chlorides on ice removal. Gonnerman, Timms and Taylor..... 107

PAVEMENT

Road—Test sections—Effect of vibration. Jackson and Kellermann..... 411

Piles—See PRECAST UNITS

Plastic flow—See LAWS AND PROPERTIES and ENGINEERING DESIGN

Portland-Puzzolan cement as used in the Bonneville spillway dam. Clark and Brown..... 183
——Disc. Bates, Carlson, Colburn, Clark and Brown, Davis. (May-June '37 Jl.).....222–1

PRECAST UNITS

Piles—Developments in foundations. Upson..... 557
Slabs—Structures in Europe. Boase..... 521
——Disc. Moreell. (Suppl. Sept.-Oct. '37 Jl.).....540–1

President's address. McMillan..... 363

Proctor, Carlton S.—Recent developments in foundation design with special references to concrete..... 541
——Disc. (Suppl. Sept.-Oct. '37 Jl.)..556–1

Properties of cements and concretes containing fly ash. Davis, Carlson, Kelly and Davis..... 577
——Disc. Derleth. (Suppl. Sept.-Oct. '37 Jl.).....612–1

Properties of job-cured concrete at early ages—Report of committee 107. Harlan H. Edwards..... 41
——Disc. Edwards, Scofield. (Jan.-Feb. '37 Jl.)..... 64–1

R

Rapid and long-time tests on reinforced concrete knee frames. Richart and Olson..... 459
——Disc. Hadley, Richart and Olson, Whitney. (Suppl. Sept.-Oct. '37 Jl.).....482–1

Recent developments in foundation design—with special reference to concrete. Proctor..... 541
——Disc. Andrew, Proctor. (Suppl. Sept.-Oct. '37 Jl.).....556–1

Recent developments in pile foundations. Upson..... 557

Reeves, A. B.—Concrete rehabilitation work on the Uncompahgre project..... 303

RESEARCH

Curing—Job conditions—Committee 107 report. Edwards..... 41
——Disc. Edwards, Scofield. (Jan.-Feb. '37 Jl.)..... 64–1

Resistance of cement to corrosive action of sodium sulphate solutions. Tuthill. 83
——Disc. Kinzie, Tuthill. (Mar.-Apr. '37 Jl.).....106–1

Richart, F. E.—Disc. Effect of plastic flow and volume changes on design—Report of Committee 313. (May-June '37 Jl.).....130–1
——Rapid and long-time tests on reinforced concrete knee frames 459
——Disc. (Suppl. Sept.-Oct. '37 Jl.)..482–1

Roelofsen, E.—Disc. Metallic aggregate in concrete floors. (Jan.-Feb. '37 Jl.)..... 28–1

S

San Francisco-Oakland Bay Bridge—Foundation design. Proctor..... 541
——Disc. Andrew, Proctor. (Suppl. Sept.-Oct. '37 Jl.).....556–1

Scofield, H. H.—Disc. Committee 107 report—Properties of job-cured concrete at early ages. (Jan.-Feb. '37 Jl.)..... 64–1

Scripture, E. W.—Disc. Evaluating fines in concrete on a bleeding test basis. (Jan.-Feb. '37 Jl.)..... 40–1
——Metallic aggregate in concrete floors..... 17

Shank, J. R.—Disc. Effect of plastic flow and volume changes on design—Report of Committee 313. (May-June '37 Jl.).....130–1

Short method for computing moments in continuous frames. Hollister..... 147
——Disc. Floris. (Mar.-Apr. '37 Jl.) 170–1

Slabs—Reinforcement—Lyse and Wernisch—See ENGINEERING DESIGN and TESTS OF MEMBERS AND COMPLETED STRUCTURES

Some comparisons of European and American concrete practice. McMillan..... 395

SPECIFICATIONS

Building—Columns—A. C. I. regulations. Willson..... 311
——Disc. Richart, Willson. (May-June '37 Jl.).....318–1
Building regulations—Standard—Committee 501 report. Stephens and Zipprodt 503
Portland cement—Federal standards. Moreell..... 435
Repair of railroad structures. Hirschthal. 251

Sprague, J. C.—Evaluating fines in concrete on a bleeding test basis..... 29

Stanton, T. E.—Disc. A study of sub-aqueous concrete. (May-June '37 Jl.) 346–1

Stephens, A. W.—Report of committee 501 Standard building code—Building regulations for reinforced concrete 503

STRUCTURAL MEMBERS

Frames—Rigid knee. Richart and Olson .. 459
—Disc. Hadley, Richart and Olson, Whitney. (Suppl. Sept.-Oct. '37 Jl.) 482–1
General—Design for flexure and compression. Whitney 483
—Disc. Johnston, Lyse, Mensch, Morris, Uqurhart, Wernisch, Whitney. (Suppl. Sept.-Oct. '37 Jl.) 498–1
—Plastic flow and volume changes—Effect on design. Morris 123
—Disc. Douglass, Richart, Shank, Whitney. (May-June '37 Jl.) 130–1
Large specimens—Drying shrinkage of. Carlson 327

Study of reinforcement in concrete slabs. Lyse and Wernisch 1
—Disc. Ernst, Hajnal-Konyi. (Jan. '37 Jl.) 16–1

Study of sub-aqueous concrete. Anderson 339
—Disc. Stanton, Anderson. (May-June '37 Jl.) 346–1

Study of the column chapter of the A. C. I. building regulations for reinforced concrete. Willson 311
—Disc. Richart, Willson. (May-June '37 Jl.) 318–1

SURFACES

Floor—Metallic aggregate in. Scripture .. 17
—Disc. Roelofsen. (Jan.-Feb. '37 Jl.) 28–1
Pavement—Scaling due to calcium and sodium chlorides. Gonnerman, Timms and Taylor 107

T

Taylor, T. G.—Effect of calcium and sodium chlorides on concrete when used for ice removal 107

TESTS AND FIELD CONTROL

Bleeding test—Evaluating fines. Sprague . 29
—Disc. Scripture, Sprague. (Jan.-Feb. '37 Jl.) 40–1
Cylinders—Under-water concrete. Anderson 339
—Disc. Stanton, Anderson. (May-June '37 Jl.) 346–1
Specimens — Job-cured — Committee 107 report. Edwards 41
—Disc. Edwards, Scofield. (Jan.-Feb. '37 Jl.) 64–1

TESTS OF MEMBERS AND COMPLETED STRUCTURES

Frames—Rigid knee. Richart and Olson. 459
—Disc. Hadley, Richart and Olson, Whitney. (Suppl. Sept.-Oct. '37 Jl.) 482–1
Road slabs—Vibrated. Jackson and Kellermann 411
Slabs—Floor—Abrasion of metallic aggregate. Scripture 17
—Disc. Roelofsen. (Jan.-Feb. '37 Jl.) 28–1
—Reinforcement studies. Lyse and Wernisch 1
—Disc. Ernst, Hajnal-Konyi. (Jan.-Feb. '37 Jl.) 16–1

Timms, A. G.—Effect of calcium and sodium chlorides on concrete when used for ice removal 107

Treated concrete—Fundamentals of Freyssinet's method. Boase 521
—Disc. Morcell. (Suppl. Sept.-Oct. '37 Jl.) 540–1

Tremie concrete—See CONSTRUCTION, DETAILS, METHODS AND COSTS

Tuthill, Lewis H.—Resistance of cement to corrosive action of sodium sulphate solutions 83
—Disc. (Jan.-Feb. '37 Jl.) 106–1

U

Upson, Maxwell M.—Recent developments in pile foundations 557

Urquhart, L. C.—Disc. Design of reinforced concrete members under flexure or combined flexure and direct compression. (Suppl. Sept.-Oct. '37 Jl.) 498–1

V

Vibration—Effect on strength and uniformity. Jackson and Kellermann 411

W

Waterproofing—See MATERIALS AND TESTS

Wernisch, George R.—A study of reinforcement in concrete slabs 1
—Disc. Design of reinforced concrete members under flexure or combined flexure and direct compression. (Suppl. Sept.-Oct. '37 Jl.) 498–1

Whitney, Charles S.—Design of reinforced concrete members under flexure or combined flexure and direct compression..... 483
—Disc. (Suppl. Sept.-Oct. '37 Jl.) . 498-1

—Disc. Effect of plastic flow and volume changes on design—Report of Committee 313. (May-June '37 Jl.) . 130-1

—Disc. Rapid and long-time tests on reinforced concrete knee - frames. (Suppl. Sept.-Oct. '37 Jl.)..... 481-1

Wilk, Benjamin—Report of Committee 710—High early strength cement in concrete masonry manufacture..... 499

Willson, C. A.—A study of the column chapter of the A. C. I. building regulations for reinforced concrete..... 311

Y

Young, R. B.—Concrete: Its maintenance and repair..... 367

Z

Zipprodt, R. B.—Report of committee 501—Standard building code—Building regulations for reinforced concrete..... 503

INDEX

CURRENT REVIEWS

From JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Vol. 8, Sept.-Oct. 1936 to May-June 1937.

For information as to indexing methods see Proceedings Index, p. 645

A

Auscultation of precast piles. Lacombe. *Le Genie Civil* 350
Autogenous healing. Loving. *Bul. Am. Conc. Pipe Assn* 510

B

Boulder Dam—Grouting of tunnels. Minear. *Civil Engineering* 356

BRIDGES

Arch—Interaction between rib and superstructure. Newmark. *Am. Soc. C. E. Proc.* 172
—French design technique. Vallette and Esquillan. *Annales de L'Institut Technique du Batiment et des Travaux Publics* 632
Flooring—Various types. Waddell. *Proc. Am. Soc. C. E.* 507
Highway—Approach span—Self-supporting reinforcing. *Eng. News-Record* 506
Rigid frame—Loup river power canal. Nicolet. *Eng. News-Record* 72

BUILDINGS

Arena—shell proof. Tedesko. *Eng. News-Record* 633
Miscellaneous—Models—Wind pressure experiments. Irminger and Nokkentved. *Ingeniowidenskatelige Skrifter* 348
Schools—Cores for control specimens. *Southwest Builder and Contractor* 70

C

Calcium ferrite hydrates. Hoffmann 177
Circular track test. Rothgery. *Rock Products* 347
Coefficient of expansion—See LAWS AND PROPERTIES—Thermal expansion.
Cold weather concreting—See also CONSTRUCTION—DETAILS, METHODS AND COSTS

CONSTRUCTION—DETAILS, METHODS AND COSTS

Bridge—Arch—Multiple span. Valenton. *Annales des Ponts et Chausseés* 174
Bridge floors. Waddell. *Proc. Am. Soc. C. E.* 507
—Self-supporting reinforcing. *Eng. News-Record* 506
Cold weather concreting. Goertz. *Beton u. Eisen* 629
Cold weather operations—Electrical superficial heating. Kunz, Fontanellaz and Haller. *Le Genie Civil* 352
Dam—Arch type—Mount Bold—Australia. Cartledge, Johnson, Campbell. *Jl. of the Inst. of Engrs. Australia* 514
—Grouting foundation. Werner. *Civil Engineering* 78
—Grouting foundation—Boulder. Jones and Minear. *Civil Engineering* 359
—Grouting joints. Hays. *Civil Engineering* 512
—Waterproof blanket on masonry. Grelot and Chalos. *Les Annales des Ponts et Chausseés* 73
Grouting—Oil well cementing—Effect of pressure. Meyers. *Rock Products* 348
Hot weather concreting. *Eng. News-Record* 627
House walls in one day—Vacuum process of dewatering. *Eng. News-Record* 629
Maintenance—Stain removal. Baumgarten. *Concrete Building and Concrete Products* 636
Pavement—Beam action in swamps. *Concrete Highways* 175
—Carriage ways in Germany. Dittrich .. 179
—Dummy joints. Bauman. *Highway Research Abstracts* 639
—Highways—German. Gutmann. *Eng. News-Record* 69
—Placing—Vibrated container. Straddling. *Roads and Road Construction* 174
—Road—Reconstruction. Beck. *Eng. News-Record* 506
—Vibration. Boyd. *Highway Research Abstracts* 640
Piles—Reinforced precast—Auscultations of. Lacombe. *Le Genie Civil* 350
Pipe, poles—Rapid hardening of. *Concrete Building and Concrete Products; Cement and Cement Manufacture* 66
Pipe—Record size. *Eng. News-Record* 66
Placing—Equipment used. Hands. *Rock Products* 66

Placing—Tunnel lining—Heating aggregates. *Eng. News-Record*..... 505
—Vibration in practice. Hollensleben. *Beton u. Eisen*..... 508
Roof — Parabolic arch — Retaining wall counterforts for. *Eng. News-Record*... 506
—Thin shell—Barrel type. Tedesko. *Eng. News-Record*..... 633
Siphon—Curing in desert heat. *Contractors and Engineers Monthly*..... 507
Tanks—Oil storage—For light oils. Harboe. *Beton u. Eisen*..... 630
Tunnel—Grouting at Boulder Dam. Minear. *Civil Engineering*..... 356
—Grouting at high pressure. Rankin. *Eng. News-Record*..... 69
—Heating aggregates. *Eng. News-Record* 505
—Placing—Pumps. Roberts. *Eng. News-Record*..... 67
—Placing machines—"guns." *Eng. News-Record*..... 67

CURING

Desert heat conditions. *Contractors and Engineers Monthly*..... 507

D

DAMS

Arch type—Mount Bold—Australia. Cartledge, Johnson, Campbell. *Jl. of the Inst. of Engrs. Australia*..... 514
Boulder—Grouting construction joints. Hays. *Civil Engineering*..... 512
—Grouting foundation. Jones and Minear. *Civil Engineering*..... 359
—Grouting foundation. Werner. *Civil Engineering*..... 78
Imperial—Colorado River—Hot weather operations. *Eng. News-Record*..... 627
Masonry—Waterproofing methods. Grelot and Chalos. *Les Annales des Ponts et Chaussees*..... 73
Deval abrasion test. Rothgery. *Rock Products* 347

DURABILITY

Disintegration—Acid action in Los Angeles sewer. *Eng. News-Record*..... 626
—Effect of alkali—Chlorides. Hundeshagen. *Zement*..... 630
—Effect of rusting reinforcement. Abeles. *Zement*..... 627

E

ENGINEERING DESIGN

Arch—Interaction between rib and superstructure. Newmark. *Am. Soc. C. E. Proc.*..... 172
Beams — Centrifugally spun — Austrian specifications. Emperger. *Beton u. Eisen*..... 634
—Safety factor in. Saliger. *Beton u Eisen*..... 173
—Two walled trusses—Restraining moments. Bortsch. *Beton u. Eisen*..... 628
—Value of "n". Emperger. *Beton u Eisen*..... 173
Bridge—Arch—Multiple span. Valenton. *Annales des Ponts et Chaussees*..... 174
Bridge floors—Monolithic with beams and girders. Granholm. *Betong*..... 509
Bridge—Large—French technique. Vallette and Esquillan. *Annales de L'Institute Technique du Batiment et des Travaux Publics*..... 632
—Rigid frame—Loup river power canal. Nicolet. *Eng. News-Record*..... 72
Buildings—Wind pressure experiments. Irminger and Nokkentved. *Ingeniorvidenskatelige Skrifter*..... 348
Columns—Eccentrically loaded. Royen. *Teknisk Tidskrift*..... 510
Foundations—Soil investigations. Loos. *Beton u. Eisen*..... 627
General—Curve of intrinsic resistance. Chalos. *Les Annales des Ponts et Chaussees*..... 75
—Future problems discussed. *Beton u. Eisen*..... 641
Pipe joint—Outfall sewer. *Eng. News-Record*..... 506
Rigid frame—Auditorium roof girder. Wright. *Eng. News-Record*..... 628
—analysis—Fixed end moments. Grinter. *Am. Soc. C. E. Proc.*..... 171
Slabs and beams—Interaction of. Kolm. *Betong*..... 68

ENGINEERING DESIGN

Tanks—Oil storage—For light oils. Harboe. *Beton u. Eisen*..... 630
Walls—Eccentrically loaded. Royen. *Teknisk Tidskrift*..... 510

F

FAILURES

Building collapse—Fire damage—Venlo market. De Boer. *Beton u. Eisen*.... 638
Fire damage to concrete building—Venlo market. De Boer. *Beton u. Eisen*..... 638
Foaming Agents—In light weight concrete. Jermalenko and Abramtschok. *Zement*... 507

L

LAWS AND PROPERTIES

Durability—Chemical deterioration. Lea. *Cement and Cement Manufacture*..... 77
Elasticity, shrinkage and strength—Effect of kind of cement. Weise. *Zement*.... 505

General—Investigation into cement concentration law. Lyse. *Jl. of the Franklin Institute*..... 74
Healing—Autogenous. Loving. *Bul. Am. Conc. Pipe Assn*..... 510
Heat generation—Maximum temperatures. Whyte. *Eng. News-Record*..... 66
Insulation—Against sound. Lindman. *Betong*..... 172
Light weight—Foamed slag concrete. *Building Research Bulletin*..... 355
——Foaming agents. Jermalenko and Abramtschok. *Zement*..... 507
Resistance—to acid action—Los Angeles sewer. *Eng. News-Record*..... 626
——to aggressive solutions. Steopoe. *Tonindustrie-Zeitung*..... 69
——to chemical attack—Effect of lime content. Rodt. *Zement*..... 68
——to oil—Coatings for. Grigorjeff and Bargtell. *Tonindustrie-Zeitung*..... 626
——to sulfate — Accelerated determination. Larmour, McMaster, Frei. *Rock Products*..... 71
——to sulfate waters. Jordt. *Tonindustrie-Zeitung*..... 65
——to sulfate waters—Determination of. Stenzel. *Industrial and Eng. Chemistry* 72
Shrinkage—Effect on durability. Abeles. *Zement*..... 627
——In road—Determination of. Graf..... 70
——Tests on. Shank. *Eng. Experiment Station News*..... 351
Strength—Alumina and slag mixtures. Vuillemin. *Le Genie Civil*..... 81
——Effect of carbonation of unhydrated cement. Bonnell. *Cement and Cement Manufacture*..... 180
——Effect of composition. Emperger and Kugi. *Austrian Committee for Reinforced Concrete*..... 176
——Effect of puzzolan content. Poulsen. *Ingenioren*..... 74
——Effect of vibration amplitude. Glanville. *The Structural Engineer*..... 636
——Relation between tension and compression. Feret. *La Revue des Materiaux de Const. et de Travaux Publics*..... 515
——Relation to other qualities. *Zementverlog*..... 176
——Variation with proportions of mix. Soroker and Desov. *Zement*..... 510
Thermal expansion—Highway pavement. Guttman and Seidel. *Beton u. Eisen*... 635
Volume changes—Effect of storage. Kayser and Buth. *Beton u. Eisen*..... 73
——Shrinkless and expanding cements. Lossier. *Le Genie Civil*..... 357
Workability—Effect of composition. Emperger and Kugi. *Austrian Committee for Reinforced Concrete*..... 176
——New method of measuring. Faury and Lamare. *Annales de L'Institut Technique du Batiment et des Travaux Publics* 627
Los Angeles rattler test. Rothgery. *Rock Products*..... 347

M

MASONRY UNITS

Brick — Flint-lime — Manufacture. *Concrete Building & Concrete Products*..... 508
Shingles—Manufacture—absestos. Itschner. *Cement and Cement Manufacture*.. 72

MATERIALS AND TESTS

Admixtures—Barium carbonate. Jordt. *Tonindustrie-Zeitung*..... 65
——For water and oil resistance. Grigorjeff and Bargtell. *Tonindustrie Zeitung*.... 626
——Method of determining proportions. Grengg and Schmolzer. *Tonindustrie-Zeitung*..... 172
——Resistance to aggressive agents. Stoepoe. *Zement*..... 628
Aggregates — and water—Proposed specifications. *Annales de L'Institut du Batimet et des Travaux Publics*..... 76
——Light weight in building. McLean. *Eng. News-Record*..... 629
——Moisture control. *Road Research Technical Paper No. 4*..... 511
——Preparation of. Hands. *Rock Products* 66
——Stone sand. Goldbeck. *Tech. Bul. No. 10, Nat'l. Crushed Stone Assn*..... 178
——Test methods compared. Rothgery. *Rock Products*..... 347
——Void calculator. Heinz. *Rock Products* 66
Alkali—Chlorides—Action on cement and concrete. Hundeshagen. *Zement*..... 630
Cements—Alumina and slag mixtures. Vuillemin. *Le Genie Civil*..... 81
——Asphalt—slime. Colberg. *Beton u. Eisen*..... 632
——Calcium oxide and magnesia. Hornke. *Tonindustrie-Zeitung*..... 626
——Chemistry problems. Kuhl. *Zement*.. 65
——Degree of burning and clinker quality. Muszgnug. *Zement*..... 509
——Disadvantages of present specifications. Dykherhoff. *Zement*..... 65
——Effect of heat of hydration. Steopoe. *Tonindustrie-Zeitung*..... 625
——Effect of heat of hydration on quality. *Zement*..... 625
——Effect of sodium chloride. Witterkindt. *Tonindustrie-Zeitung*..... 172
——Effect of storage. *Le Genie Civil*..... 354
——Effect of sulfates and sulfides in clinker. *Tonindustrie-Zeitung*..... 347
——Effect of vibration on strength. Glanville. *The Structural Engineer*..... 636

—Effect on properties of concrete. Weise. *Zement*..... 505
—For road construction. Schwiete. *Zement*..... 508
—German methods. *Zementverlog*..... 176
—Grain size of. *Tonindustrie-Zeitung*.... 67
—High early strength—Arch roof construction. *Eng. News-Record*..... 506
—Hot—Effect of. Kruger. *Zement*..... 506
—Lime content and resistance to chemical attack. Rodt. *Zement*..... 68
—Manufacture—Charging tube mills. Steiner. *Tonindustrie-Zeitung*..... 347
—Clinker burning methods. Lipinski. *Tonindustrie Zeitung*..... 347
—Double hammer crusher. Korte. *Zement*..... 66
—German association. *Zementverlog*.. 176
—German innovation. Jordan. *Zement*..... 505
—History of industry in India. *Spec. Pub. Associated Cement Companies Ltd*..... 349
—Kiln ring. Yoshii. *Zement*..... 505
—Quality of clinker. Anselm. *Zement* 175
—Thermochemical principles. Gronow. *Zement*..... 67
—Microscopic study of. Stach. *Tonindustrie-Zeitung*..... 626
—Oil well—Effect of pressure. Meyers. *Rock Products*..... 348
—Portland-puzzolan. Poulsen. *Ingeniören*..... 74
—Resistance to chemical attack. Lea. *Cement and Cement Manufacture*..... 77
—Special—European investigations. Sevieri. *Tonindustrie-Zeitung*..... 625
—Unhydrated—Carbonation of. Bonnell. *Cement and Cement Manufacture*..... 180
—Shrinkless and expanding. Lossier. *Le Genie Civil*..... 357
—Slag—Sulfides in. Sevieri. *Tonindustrie-Zeitung*..... 171
—Special. Jennings. *Cem. & Cem. Manufacture*..... 351
—Special for mass concrete. Savage. *Bureau of Reclamation*..... 69
—Special for mass concrete. Joint subcommittee on special cements..... 511
—Specific surface of. Hornke. *Tonindustrie-Zeitung*..... 348
—Specific surfaces of. Von Gronow. *Tonindustrie-Zeitung*..... 171
—Systematic designation of. Solacolu. *Zement*..... 70
Portland cement—Calcium ferrite hydrates. Hoffman..... 177
Portland cement—Catalysis and dispersion. Kennedy. *Industrial and Engineering Chemistry*..... 71
—Clinker research. Kuhl. *Tonindustrie-Zeitung*..... 171
—Determining sulfate resistance. Stenzel. *Industrial and Engineering Chemistry*... 72
—German industry. Czernin. *Tonindustrie-Zeitung*..... 67
—Manufacture — Clinker compounds. Schwiete. *Tonindustrie-Zeitung*..... 631
—Clinker cooling. Lipinski. *Tonindustrie-Zeitung*..... 65
—Granulating raw feed. Schrim. *Tonindustrie-Zeitung*..... 631
—Simultaneously with iron. Callado. *Tonindustrie-Zeitung*..... 68
—Reconditioning after deterioration. *Concrete*..... 637
—Statistical study of tension. Leavitt. *Paper No. 18, University of Maine*..... 358
—Sulfate resistance—Determination. Larmour, McMaster, Frei. *Rock Products*. 71
Powdered rock—Specific surfaces of. Von Gronow. *Tonindustrie-Zeitung*..... 171
Puzzolana—Artificial. Czerin. *Tonindustrie Zeitung*..... 348
Slag and alumina mixtures. Vuileemin. *Le Genie Civil*..... 81
Slags—Basic—Sulfides in. Sevieri. *Tonindustrie-Zeitung*..... 171
—Foamed—Light weight concrete. *Building Research Bulletin*..... 355
Sodium chloride—Effect on cement. Witterkindt. *Tonindustrie-Zeitung*..... 172

MIXTURES

Proportions—Analysis of solid masses. Grengg and Schmolzer. *Tonindustrie-Zeitung*..... 172
—Determination of cement content. Gonnell. *Zement*..... 65
—Determination in hardened mortar. Stoepel. *Tonindustrie-Zeitung*..... 625
Design—Economy basis. Lyse. *Jl. of the Franklin Institute*..... 74
—Methods of determining composition compared. Feret. *Le Genie Civil*..... 353
—Specifications for—Proposed German. *Tonindustrie-Zeitung*..... 347

P

PAVEMENT

Carriageways—Germany. Dittrich..... 179
Highways—Beam action in swamps. *Concrete Highways*..... 175
—Life expectancy of. Allen. *Highway Research Board Abstracts*..... 79
—Expansion joints—Function of. Guttmann and Seidel. *Beton u. Eisen*..... 635
—Reconstruction methods. Beck. *Eng. News-Record*..... 506
Road—Experiments with various surfaces. Moyer and Tillapaugh. *Proc. Highway Research Board*..... 513
—Shrinkage in—Determination. Graf. *Zement*..... 70

—Surfacing — Reinforced. Velle. *Schweizer, Zeit, Strassenwessen* 173

PRECAST UNITS

General—Production methods. *Cement and Cement Manufacture, Concrete Bldg. & Concrete Products* 349
Pipe — Centrifugal — Manufacture. *Concrete Building and Concrete Products* 508
—poles—Rapid hardening. *Concrete Building and Concrete Products; Cement and Cement Manufacture* 66
—Record size. *Eng. News-Record* 66
Shingles—Asbestos—Effect of temperature. Itschner. *Cement and Cement Manufacture* 72

R

RESEARCH

Cement—Clinker investigations. Kuhl. *Zement* 65
—Developments in Europe. Sevieri. *Tonindustrie-Zeitung* 625
Concrete Developments and processes in Europe. Sevieri. *Tonindustrie Zeitung*. 625
Portland cement—Manufacture—Clinker problems. Kuhl. *Tonindustrie-Zeitung* 171

S

Sound insulation—European views. Lindman. *Betong* 172

SPECIFICATIONS

Aggregates and water—Proposed in France. *Annales de L'Institat du Batimet et des Travaux Publics* 76
Beams — Centrifugally spun — Austrian standard. Emperger. *Beton u. Eisen*.. 634
Cement—Disadvantages of tests methods. Dyckerhoff. *Zement* 65
—Special—Comparison of U. S. and England. *Report to Joint Sub-Committee on Special Cements of Inst. Research Com. and British Com. on Large Dams* 511
Mix and cement content—Proposed German. *Tonindustrie-Zeitung* 347
Plain and reinforced concrete—German—Criticism of. *Beton u. Eisen* 641
Reinforced concrete—German—Proposed revisions of. Emperger. *Beton u. Eisen* 633

STRUCTURAL MEMBERS

Beams—Centrifugally spun. Emperger. *Beton u. Eisen* 634
Girders—Hollow—Rigid frames. Wright. *Eng. News-Record* 628

SURFACES

Cleaning—Stain removal. Baumgarten. *Concrete Building and Concrete Products* 636
Road—Effect on cost of auto operation. Moyer and Tillapaugh. *Proc. Highway Research Board* 513
—Reinforced. Velle. *Schweizer, Zeit, Strassenwesen* 173

T

TESTS AND FIELD CONTROL

Apparatus—Vibrated. Stradling. *Roads and Road Construction* 174
Cores—Replacing cylinders. *Southwest Builder and Contractor* 70
Cube specimens — Winter concreting. Goertz. *Beton u. Eisen* 629
Methods—Moisture control. *Road Research Technical Paper No. 4* 511

TESTS OF MEMBERS AND COMPLETED STRUCTURES

Arches—Tenperature in ribs. Whyte. *Eng. News-Record* 66
Bridges—Slab and beam interaction. Kolm. *Betong* 68
Columns—Steel encased in concrete. Stang, Whittemore, Parsons. *Jl. of Research, Nat. Bureau of Standards* 178
Pavement slabs—Freezing and thawing cycles. Allen. *Highway Research Board Abstracts* 79
Slabs—Experiments at Baku. Steuermann. *Beton u. Eisen* 349

V

Vibration—Concrete pavement construction. Boyd. *Highway Research Abstracts* 640
—Influence of amplitude on strength. Glanville. *The Structural Engineer* 636
—Precast unit production. *Cement and Cement Manufacture, Concrete Bldg. and Concrete Products* 349
Vibrators in practice. Hollensleben. *Beton u. Eisen* 508